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ORDINARY MEETING.

21 February, 1939.

MAURICE FITZGERALD WILSON, Vice-President, in the Chair.

The Council reported that they had recently transferred to the class of

Members.

Haddon Clifford Adams, M.C., M.A. (*Cantab.*).
Abdel Aziz Ahmed Bey, D.Sc., Ph.D. (*Birmingham*).
Charles Dimond Conway Braine, B.Sc. (*Birmingham*).
Herbert Wallis Coales, O.B.E., M.C.
Thomas Henry Longstaff.
Humphrey Davys Manning, B.Sc. (*Birmingham*).

And had admitted as

Students.

Francis Prescott Charles George Atkinson.
John Harold Stirling Bridger.
Jesse Vernon Brightmore.
Richard Frank Bullivant.
Keith Clarence Burdon.
Lionel Studd Buxton.
Alwyn Paul Leonard Carr.
Peter Francis Carter, B.Sc. (Eng.) (*Lond.*).
Gordon Barker Chapman.
Arthur William Cherry.
Robert Norman Clark.
Michael Arthur Clinton.
Noel James Cochrane.
Edmund Collins.
David Alexander Fleming Colville, B.Sc. (*Edin.*).
George Maxwell Cornfield.
Leonard Frederick Crossley.
Colin Massey Dennes.
Herbert Raymond Dodd.
James Graham Eakin.
William Leamington Elsby.
Lenox Paton Cleland Figgis.
John Frederick Fone, B.Sc. (Eng.) (*Lond.*).
Reginald James Garvie.
Charles Melvin Gibbs.
Naim Odeh Goussous.
Vincent Louis Granger, B.Sc. (*Cape Town*).
William Henry Green, B.Sc. (*Birmingham*).
Eric Arthur Sinclair Guy.
William Brown Harris.
Colin Frederick Ian Harrison.
Charles George Henderson.
Barton Frank Higgs.
George Horne.
David Oliver Humphreys.
Frank Victor Jago.
Edward Arthur Labrum.
Arnold Lackie.

James Lang.
Dudley Roger Lester.
Eric Ignatz Loewy.
Kenneth Longley.
Imaduddin Mohammad Khan Malik, B.E. (*Bombay*).
James Hunter Moffat.
Thomas Redding Montague.
Thomas Morrison.
Claude Murison.
Balasubramaina Iyer Natarajan.
Edward Rhys Newens.
Nanda Nimalasuriya.
Frederick Norman Pannell, B.Sc. (Eng.) (*Lond.*).
John Shepard Payne.
John Aylmer Fitzgerald Leaver Power.
Martin Charles Privett.
Kenneth Dunbar Rhodes, B.Sc. (Eng.) (*Lond.*).
Dewi Ieuan Richards, B.Sc. (*Birmingham*).
Alun Rogers.
Jack Geoffrey Rogers.
Yusuf Mohamed Said.
Kalu Javhermal Shahaney.
John Thompson Shepherd.
Ambika Saran Singh.
John Malfaison Smales.
Geoffrey Stead.
Angus Anthony Gentelles Sutton.
Roger Thornton.
Julian Corbett Turnbull, B.A. (*Cantab.*).
Eric William Kevin Walton.
David Nigel Ward.
Desmond Milton Whitehouse.
John Alan Williams.
John Williamson, B.Sc.(*Glas.*).
Frank Reginald Woods.

The following Paper was submitted for discussion, and, on the motion of the Chairman, the thanks of the Institution were accorded to the Authors.

Paper No. 5197.

"The Storstrøm Bridge." †

By GUY ANSON MAUNSELL, B.Sc. (Eng.), M. Inst. C.E., and JOHN FREEMAN PAIN, M.C., B.Sc. (Eng.), Assoc. M. Inst. C.E.

(*Abridged.**)

TABLE OF CONTENTS.

INTRODUCTION.

Site and Character of Works.

The Storstrøm and Masnedsund bridges not only provide a road and rail connexion without any break between the principal Danish island of Zealand, on which Copenhagen stands, and the island of Falster to the south of Zealand, but they also permit through traffic to pass from Copenhagen to the continent of Europe without any further break than that occasioned by the train-ferry passage of 25 miles in length between Gedser in Falster

† Correspondence on this Paper can be accepted until the 15th July, 1939.—SEC. INST. C.E.

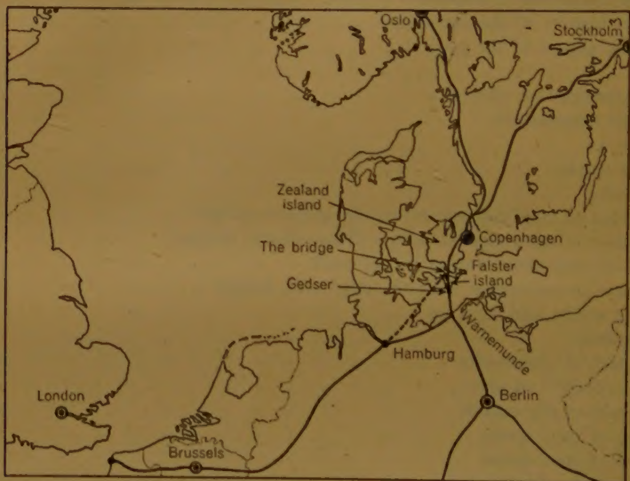
* The MS. and illustrations may be seen in the Institution Library.—SEC. INST. C.E.

and Warnemünde in Germany (*Fig. 1*). The route has a further importanc as providing a direct and convenient passage between Central Europe an Norway and Sweden.

The distance between the islands of Zealand and Falster measure along the centre line of the selected route is a little over 6 kilometre (about 3¾ miles), 2½ kilometres of which lies upon a small intermediat island called Masnedø.

Although there is a narrow sea channel between Masnedø and Zealan called the Masnedsund, the main channel to be bridged is called th Storstrøm or great stream, and lies between the islands of Masnedø an Falster. It is nearly 4 kilometres (2½ miles) wide, and is used by abou 15,000 ships annually.

Fig. 1.



KEY PLAN OF SITE.

The general scheme of the viaduct (Figs. 2, Plate 1) consists of a low level bridge with six short spans, one of which is a bascule opening-spa over the Masnedsund, followed by a long embankment on a rising gradien curved like the letter S across the island of Masnedø leading up to th high-level bridge with fifty large spans, including three navigation-span about mid-way across the channel. The central navigation-span has clear opening of nearly 400 feet and gives a headroom above mean sea level of over 85 feet. The viaduct and bridges carry a single line of railwa on one side and a roadway for two lanes of traffic on the other side.

Fig. 3, Plate 1 gives a diagrammatic view of the layout of the viaduc and bridges.

Before the bridge was built there existed across the Masnedsund

narrow single-track railway-bridge, whilst the main crossing of the Storstrøm used to be effected by means of train-ferries, a form of transport in which the Danes have long excelled.

Previous History of the Enterprise.

In 1887 the Danish State Railways had already worked out a plan for a low-level bridge with a central swing-span in the Storstrøm, and similar schemes were again under discussion in 1893 and 1903. However, in 1910 a vote in the Danish Parliament showed equal numbers for and against the project, and failing a clear majority in favour no progress was made.

In 1914 designs for a single-line tube railway-tunnel were evolved but the intervention of the European war prevented further progress being made.

In recent years it has become increasingly evident that a satisfactory design would have to include facilities for road transport, and ultimately, in 1931, the Danish State Railways produced a scheme for a combined road and railway bridge, which, in its location and outline, corresponded fairly closely with the final project as it has been built.

The acceptance by the Danish government of the British design and offer to construct such a bridge was largely due to the exertions of Mr. Charles Mitchell, M. Inst. C.E., at that time chairman of Messrs Dorman, Long & Co., Ltd., and of Mr. James Osborne, Assoc. M. Inst. C.E., Manager of their Bridge Department.

Economic Aspect of the Project.

It is interesting to inquire whether or not large bridge-projects are economically sound, but it is very difficult to assess economic values in terms of figures and is especially difficult in a case such as this where the rail- and road-traffic is combined, and where many imponderable factors have to be taken into consideration, such as the stimulus to tourist-traffic and business activity resulting from improved facilities, the military value of the works, and the possibility that traffic may be diverted from rail to road.

The economic value of this particular project consists of two parts: firstly, its economic value as a railway connexion, and secondly, its value as a roadway connexion.

In assessing the economic value of the railway through-connexion, the Danish State Railways estimated that after making certain allowances, the abolition of the train-ferry service would effect an annual saving of a little over 1 million kroner (£45,000), which, capitalized at 5 per cent., represented a figure of just over 20 million kroner (£900,000).

The abolition of the ferry service made it necessary, however, to write off the amount (2·9 million kroner) by which the book value of the ferry-boats, piers, etc. exceeded their break-up value, and also to capitalize and deduct 0·9 million kroner in respect of increased charges for pensions to discharged ferrymen and others, and a further 0·9 million kroner in respect of the interest payable on the construction money outstanding during the construction period.

This left 15·7 million kroner, or approximately £700,000, as being the amount which the Danish State Railways felt they could justifiably contribute towards the cost of the project. The whole cost of the bridges and viaduct amounted to approximately £1,380,000, so that the Railways' contribution was about half of the total, and the remaining £680,000 has to be allocated against the roadway connexion.

According to a Danish law the interest and amortization of this £680,000 (or whatever the balance actually may be) is to be effected by means of a tax of 1 øre per litre ($\frac{1}{3}d.$ per gallon) on all motor spirit imported into or manufactured in Denmark. After the capital sum has been paid off in this way the tax is to remain in force until a further capital fund accumulates, sufficient to meet maintenance charges in perpetuity, which are expected to be from 50 to 60 thousand kroner (about £3,000) per annum.

Thus, it has been decided to distribute the cost over all motor vehicles in Denmark rather than to impose a toll-charge (which might have amounted to the equivalent of one shilling and sixpence) only on vehicles actually crossing by the bridge. This procedure has the advantage of possessing an encouraging rather than a restraining effect on the volume of traffic using the bridge.

In the year 1932 the number of motor-cars crossing by ferry was estimated to be 45,000, and in 1936 the number had increased to about 100,000.

During the twelve months after the bridge was first opened, that is from the 1st October, 1937 to the 1st October, 1938, 383,500 motor vehicles made use of the bridge.

In the year before the bridge was opened, 480,000 passengers and 370,000 tons of goods crossed the Storstrøm by rail-ferry and there was a considerable further quantity of traffic which used to be carried by a privately-owned ferry plying between Masnedsund and Falster.

The figures quoted above may be of interest as showing that in a small agricultural country like Denmark, where the total population is under 4 millions and where the traffic is by no means heavy, great and costly works such as these are being undertaken with every appearance of economic justification.

The building of the bridge has been financed by means of a Danish loan for 1 million pounds sterling raised in England in 1933 and guaranteed as to interest and repayment by the Danish Government.

Costs.

The total cost of the works was as follows :—

	Kroner.	Sterling equivalent.
Storstrøm bridge	28,500,000	1,270,000
Masnedsund bridge	2,500,000	111,000
Approach railways and stations	7,000,000	311,000
Approach roadways and works	3,000,000	133,000
Total	41,000,000	£1,825,000

The above figures include the cost for the road and railway improvements on shore.

The contract signed with Messrs. Dorman, Long and Company provided as follows :—

	Kroner.	Sterling equivalent.
Bridgeworks, sterling expenditure	—	579,657
Bridgeworks, expenditure	10,657,876	474,000
Additional works expenditure	5,700,000	253,000
Total	—	£1,306,657

The difference between the actual cost and the contract figure is accounted for by the fact that the Danish State Railways supplied all the cement, timber, and steel rails free of cost to the Contractors, and themselves laid the railway track and did certain other works by direct labour.

Technical Description of Works.

Piers and Foundations.

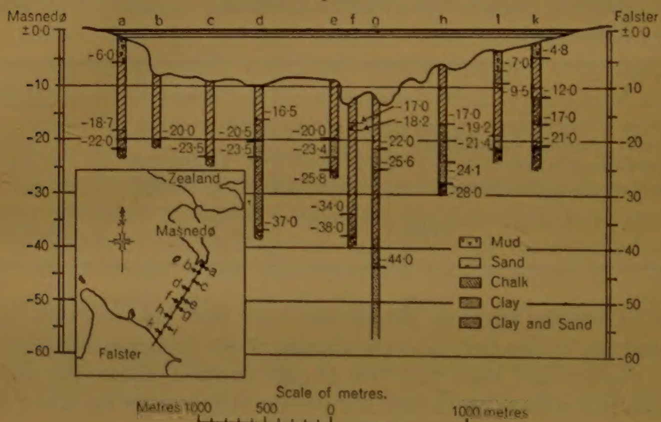
The Baltic is a tideless sea of very clear water, half salt and half fresh. A strong east wind tends to pile the water up in the western part of the Baltic (where the bridge is situated), and vice versa, so that there is sometimes a variation in water-level due to wind-action of about 3 feet above or below mean sea-level. For the same reason strong currents exist in certain channels, but the current in the Storstrøm was never more than moderate—about 3 knots.

The site of the bridge is more or less land-locked, so that heavy seas are not encountered, but in winter there is a danger of ice, and the bridge has been designed to resist the impact of pack-ice. The pack-ice occurs if a strong wind follows upon a thaw after severe frost. On such occasions ice may be piled up 15 feet high on the shores of the islands. Severe ice-condi-

tions only occur during about four winters in twenty-five, and fortunately a severe ice-winter did not happen during the construction-period.

The land bordering the site of the bridges is rather flat, and the sea is not deep, the greatest depth beneath the bridges being about 46 feet. *Fig. 4* shows the nature of the sea bottom. It consists generally of a layer of light grey-coloured glacial clay, rich in lime, from 13 to 33 feet in thickness overlying chalk. The clay was found to be rather soft in places and sometimes stiffer on top than beneath. It was fairly watertight in most places, but in some parts it contained a good deal of sand and was liable to boil up or " blow " under hydrostatic pressure from beneath. For this reason some of the foundations could not be pumped out and the excavation work and deposition of concrete foundation-slabs were carried out under

Fig. 4.



BORINGS ACROSS THE STORSTRØM.

water. In most cases, however, the foundations of piers were safely carried out in the dry.

The low-level bridge across the Masnedsund was built upon four small piers, one larger pier built to house the trunnion, counterweight, and operating mechanism of the bascule-span, and two bridge-abutments connecting the terminals of the shore-embankments.

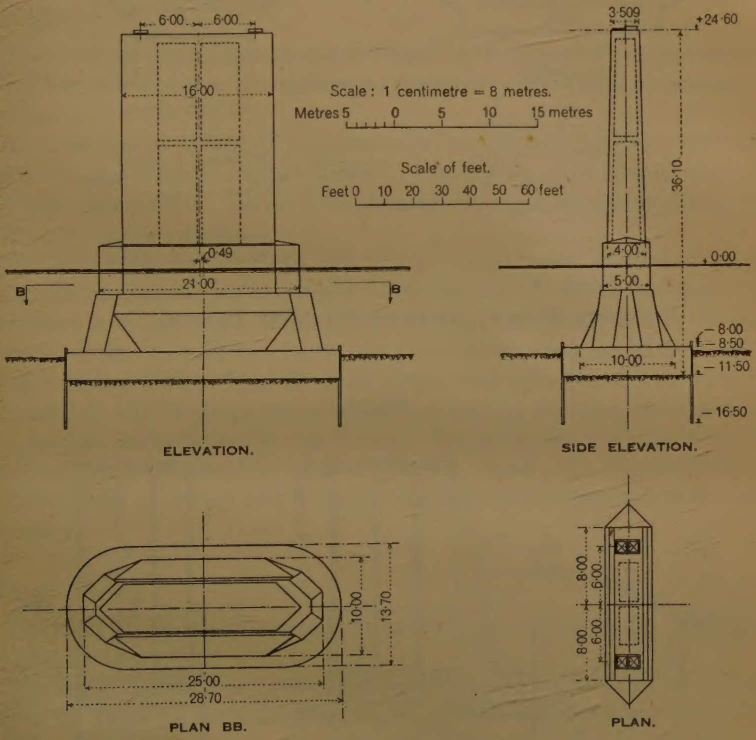
Steel sheet-pile cofferdams were driven around the pier-sites in the first instance, the material being afterwards excavated by grab, and concrete deposited in some cases in the dry and in other cases under water. One of the abutments and one of the piers were founded on timber piles which were driven and cut off just below water-level. The piles were rough round logs with the bark on, cut from native spruce.

Experience has shown that such timber is immune from decay when immersed in the Baltic water, and the cost per cubic foot driven is very

much lower than the cost of either timber or concrete piling as carried out in Great Britain.

The piers of the main Storstrøm high-level bridge were of two types, there being four very large high piers supporting the navigation-spans and forty-five other piers of varying heights and sizes. Details of these two types of piers are given in *Figs.* 5 and 6 respectively. The largest piers

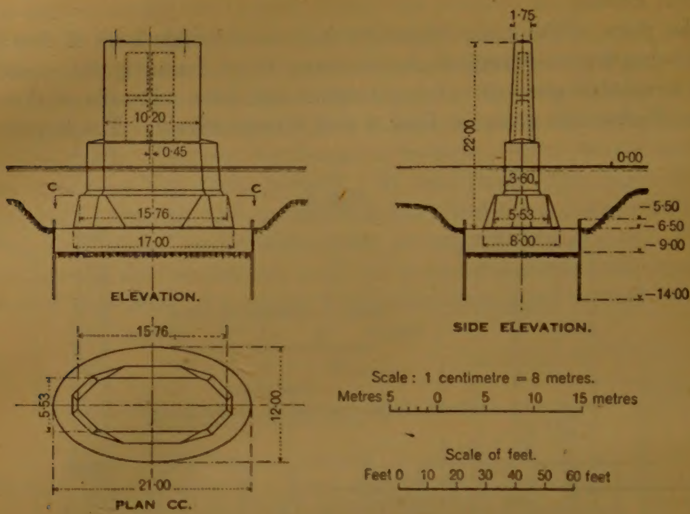
Figs. 5.



STORSTRØM BRIDGE: NAVIGATION-SPAN PIER DETAILS.

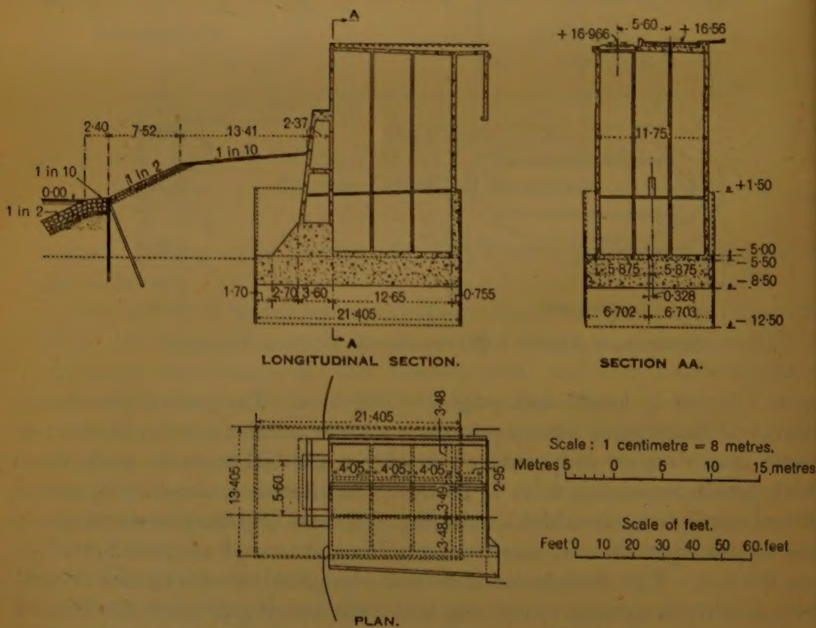
were 118 feet in height and weighed 8,000 tons. The general plan in the design of these piers was to provide a slender cellular reinforced-concrete shaft above water, supported on a solid reinforced-concrete shaft, faced with granite extending from 2½ metres (8 feet 3 inches) above to 2½ metres below water, beneath which a mass-concrete base of prismatic shape splays out to rest upon an oval mass-concrete foundation-slab situated below the sea-bottom. The foundation-slab was designed to spread the weight over a fairly large area of clay, on which the bearing pressure was limited to about 3·2 tons per square foot.

Figs. 6.



STORSTRØM BRIDGE : APPROACH-SPAN PIER DETAILS.

Figs. 7.



STORSTRØM BRIDGE : SOUTH ABUTMENT DETAILS.

The reinforcing steel provided in the cellular walls of the pier-shafts is intended mainly to counteract cracking due to shrinkage of the concrete.

The south abutment of the high-level bridge is illustrated in *Figs. 7*. These abutments had to serve the double purpose of supporting one end of the shore-span and of retaining the end of the very high approach-embankment. The construction adopted is in reinforced concrete made cellular for lightness and economy, and made monolithic for strength and stability.

Earthworks.

There was a large amount of earthwork in the approach embankments to the high-level bridge, amounting to 2,300,000 cubic yards of excavated material.

The maximum height of embankment was 57 feet 6 inches with side slopes of 1 in 1·5 for the upper 20 feet, 1 in 2 for the next 20 feet, and 1 in 2·5 for the lower part. Berms 6 feet 6 inches in width were provided at the 20-foot vertical intervals.

The water at the southern end of the bridge being shallow, the embankment was carried out to project 1,300 feet into the sea at that end. To commence with, a 13-foot depth of surface-mud was removed by grab and the area of the base of the bank was then surrounded by timber sheet-piling, 5 inches thick. Into the space inside the piling, sand was pumped and raised to 13 feet above water-level, after which a superimposed embankment was tipped out from the shore end. Stone rip-rap was tipped on the outside of the piles, and the outer slopes were faced with granite pitching up to the level of 13 feet above sea-level as a protection against wave- and ice-action.

Railway Works.

The total length of the new railway on both sides of the bridge amounted to 13 kilometres.

The rails used were flat-bottomed, weighing 45 kilogrammes per metre, welded in pairs to form lengths of 30 metres, and fastened to native beechwood sleepers of approximately 10 inches by 6 inches in section.

On the bridge itself the rails were welded together to correspond with the length of the bridge-spans, and as the expansion to be allowed for at the rail-joints was in the worst cases as much as 7 inches, scarf-joints between the metals had to be provided. Guard-rails were used on the bridge.

Roadworks.

The original road-access to the ferry on the north side used to run through narrow streets in the town of Vordingborg, and it was decided to by-pass the town with the new roadway.

The width of the carriageway in the centre of the approach road is 6·5 metres (about 21 feet 6 inches) flanked by two cycle-tracks each 1·5 metres (about 5 feet) wide, outside which are footways 0·75 metres (about 2 feet 6 inches) wide.

The carriageway is of 8-inch thick reinforced concrete, with fabric reinforcement (10·5 lb. per square yard), and was tamped in two layers by a special concrete-road-laying machine; 1-inch ballast-concrete was used for the lower layer and $\frac{3}{4}$-inch granite chips for the upper. The quantity of cement used varied between 472 and 590 lb. per cubic yard of finished concrete.

The cycle-tracks are formed in vibrated concrete laid on a sand base, the concrete being 4 inches thick on the side near the road and $2\frac{1}{2}$ inches thick on the outer side. The tracks are separated from the carriageway by a flush-laid strip of asphalt 3 inches wide. The weight of the reinforcement in the track is approximately 2 lb. per square yard. The footways are also formed of vibrated concrete slabs, 2 inches thick, laid on sand and provided with a curb.

The roadway has a camber of just over 2 inches and has a central longitudinal bitumen joint and also transverse joints about 65 feet apart, but the spacing of the latter is varied to avoid the setting up of periodic oscillation in cars travelling at speed.

The ruling gradient on the roadway is 1 in 30, the minimum vertical radius is 2,000 metres (about 6,500 feet), and the minimum horizontal radius 300 metres (about 1,000 feet). Superelevation is provided on the horizontal curves.

The roadway carried on the bridge decks (Fig. 8, Plate 1) is formed in two layers with a sheet of asphaltic material between the two. The lower layer of concrete is the structural member and contains the reinforcement. The purpose of the watertight sheet is to preserve the steel in the lower layer from corrosion which might result from surface-water seeping through the upper layer of concrete, should it deteriorate in course of time.

Description of Steel Superstructure (*Storstrom Bridge*).

Approach-Spans.

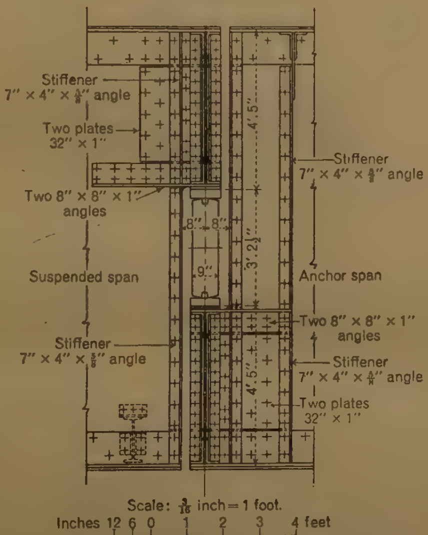
The approaches, or side-spans, are constructed as deck-type plate-girders of cantilever type (Figs. 2 and 9, Plate 1), with suspended spans of 145 feet 10 inches in alternate openings. Anchor-spans are 189 feet 7 inches centre to centre of bearings. Suspended spans are supported on 29-foot 2-inch cantilever-arms. Each span is carried by two main plate-girders 24 feet centre to centre and 12 feet $0\frac{1}{2}$ inch deep over flange-angles.

Flanges consist of 12-inch by 12-inch angles with from one to four flange-plates 27 inches wide. Web-plates are $\frac{11}{16}$ inch thick divided by a central longitudinal splice. Webs are stiffened by pairs of vertical angles on packings spaced from 35 inches to 87·5 inches apart. In addition, the

compression side of the web-plate is stiffened against buckling by short lengths of longitudinal angle midway between the flange and central splice near the centre of the anchor and suspended spans. 1-inch diameter high-tensile steel rivets in $1\frac{1}{16}$-inch holes have been used throughout the main girders.

The anchor-spans are supported on fixed and expansion bearings of cast steel upon alternate piers. Fixed bearings are of the spherical rocker type. Expansion bearings are of the single-rocker Haberkalt type, with rockers $61\frac{5}{8}$ inches high (Figs. 10, Plate 1). The suspended spans are supported at both ends upon single cast-steel rockers, $32\frac{1}{2}$ inches in diameter, bearing on flat steel seatings resting on the projecting ends of

Fig. 11.



Connexion of Suspended and Anchor Spans.

adjacent spans (*Fig. 11*). The fixed end of each suspended span is secured to the end of the cantilever-arm by a vertical pin-joint mid-way between the girders in the plane of the girder top flange, the other end being allowed to expand by means of a knuckle working between jaws (*Fig. 12*, p. 402).

The girders are connected by full lateral systems in the planes of their top and bottom flanges, built up from pairs of starred angles. Provision is made in the top lateral system to resist tractive forces.

Portal-bracing is provided in the plane of the bearings of the anchor-spans.

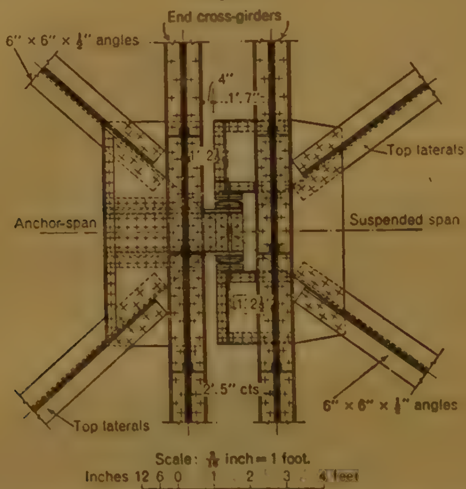
The deck is carried by plate cross girders resting on top of the main girders and cantilevered out to support the roadway (Fig. 9, Plate 1). Stringers are all of rolled joist sections. Railway-stringers are framed

into cross-girder webs and connected by continuity plates. Roadway-stringers rest on top of cross-girders to which they are connected by clips permitting a limited longitudinal movement. Rivets in the deck-system are generally ⅞ inch in diameter.

The deck is arranged to produce nearly equal moments on the two main girders, which were designed alike.

The railway track is carried on ballast in a reinforced-concrete trough resting on the stringers. The roadway is carried on a reinforced-concrete slab, from the outer edge of which the footway-slab is cantilevered without independent stringers. The deck-slab is insulated with a bitumastic

Fig. 12.



EXPANSION LATERAL CONNEXION OF SUSPENDED AND ANCHOR SPANS.

layer upon which rests a 4½-inch concrete wearing surface. The footpath is surfaced with 1-inch asphalt.

A sliding joint is provided in all stringers at both ends of the suspended span, and at the centre of the anchor-span. The main roadway expansion-joints are steel castings of the interlocking-finger type. Footway and ballast-trough expansion-joints are of sliding-plate construction.

A hand-operated painting-traveller gives access to the underside of the steelwork for the full length of each approach. Each traveller moves on a track hung below the cantilever ends of the cross girders, and consists of a framework supporting a counterbalanced rotating arm which can be turned to project below the steelwork.

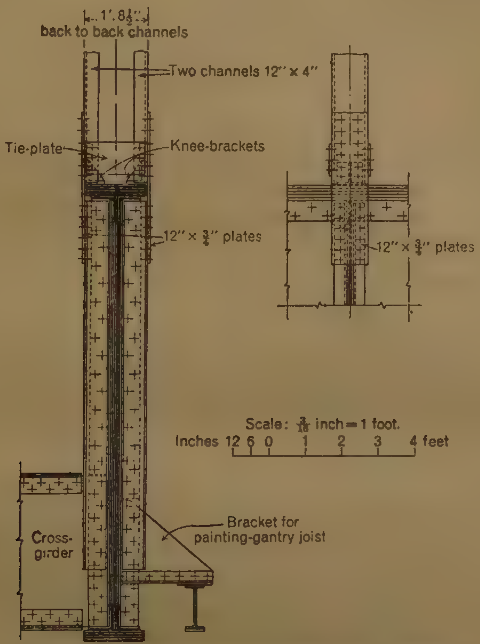
Navigation-Spans.

The three navigation-openings are spanned by tied stiffened arches of through construction, the stiffening being provided in the form of parallel

plate-girders at deck-level. The deck and polygonal arch-rib are connected by vertical hangers, and the main girders are spaced 39 feet $4\frac{1}{2}$ inches centre to centre. Sectional details of a navigation-span are given in Fig. 8, Plate 1.

The arch-ribs consist of two built-up channel-sections with flanges turned outwards, connected by a solid upper flange-plate and by a single angle-lacing. Due to the fact that the road and railway tracks lie between the stiffening girders, it was not possible to obtain equal loading on both

Figs. 13.



HANGER CONNEXION TO STIFFENING GIRDER.

girders, and the sections of the railway-side girder are therefore uniformly less than those on the roadway-side.

The arch-ribs are connected by a system of double diagonal lateral bracing without cross members, extended as far as the roadway clearance will permit. All lateral members consist of rolled-steel joists.

Portal-bracing is provided on the vertical hangers at the ends of the lateral system to transfer any lateral force from the ribs to the deck.

The longest hangers consist of a pair of battened channels. Shorter hangers are of built-up I-section. Details of a hanger connexion to the stiffening girder are given in *Figs. 13*.

The stiffening-girders, which also form the arch-ties, are designed as

plate-girders, 12 feet 0½ inch deep. The intersection point with the axis of the arch-rib is situated 4 feet $3\frac{3}{16}$ inches above the centre-line of the stiffening-girder, to assist in equalizing the stresses on the girder-flanges.

A full system of lower lateral bracing of joist-sections is provided, attached to the bottom flange of the stiffening girder.

The end cross girders are designed to resist the effect of tractive forces set up by trains crossing the bridge.

End bearings are of the cast-steel spherical-rocker type. The expansion-bearings are mounted on pairs of cast-steel segmental rollers 28 inches in diameter (Figs. 14, Plate 1).

Provision is made in the design of the end cross girders for lifting the spans on jacks in the event of any settlement of the piers.

The deck steelwork is of conventional cross-girder and stringer construction. The footway is carried on cantilever-brackets outside the main girder.

The railway deck is of open timber construction, laid direct on the stringers, alternate sleepers being cleated to the flanges. Guard-rails are employed for the full length of the navigation-spans. The road and footway decks are similar to those on the approaches.

Joints are provided in the deck and stringers at both ends and at the centre of each span. The main expansion-joints at one end of each span and other joints in the deck are of similar type to those on the approaches.

A painting-traveller is provided below the deck of each span. Inspection-walkways are provided below the arch-ribs.

Description of Steel Superstructure (Masnedsund Bridge).

Fixed Spans.

Each of the five fixed spans is carried by two simply-supported plate-girders 37 feet 10¾ inches apart. Girders are 8 feet 4½ inches deep over angles with ½-inch web-plates, and are provided with a full system of lower lateral bracing in the plane of the bottom flanges.

Cast-steel spherical rocker-type bearings are provided at both ends of each span, the expansion bearings being carried on a pair of rollers.

The deck is of similar construction to that of the navigation-spans. A section through one of the spans is given in Fig. 15, Plate 1.

Bascule-Span.

The bascule-span is carried by two main plate-girders of similar construction to those used in the fixed spans. Hollow trunnion-pins are cantilevered out from the girders and rest on bearings at the front of the pier. Heel-extensions of the main girders project across the pier and carry the main driving pinions which operate on cast-steel toothed racks mounted on the back wall of the pier. The operating machinery is mounted in a cabin on the pier, and the drive is transmitted by shafting and bevel gearing through the hollow trunnion to the driving pinion. A steel-plated

ballast-box filled with old rails grouted in position and weighing 560 tons is accommodated between the main girders at the heel of the span.

The main machinery, supplied by Sir William Arrol and Company, of Glasgow, comprises a first motion unit on one side of the bridge driven by a 70-brake-horse-power 380-volt 3-phase alternating-current motor, or alternatively by a 70-brake-horse-power 440-volt direct-current motor as a standby. Both motors were 1-hour rated, to stand 100 per cent. overload for 3 minutes. Emergency hand-operating gear is provided on the other side of the bridge. The power from these units is transmitted to a central longitudinal shaft by spur-and-bevel gearing. The torque on the rack-pinion shafts is equalized by means of a differential between them and the longitudinal shaft.

The span is designed to be fully opened against normal resistance in 75 seconds.

The roadway deck of the bascule-span is of Danish oak, surfaced with a mat of hemp rope-fabric plastered with hot asphalt into which a grit wearing-surface is rolled.

Résumé of Statistics relating to the Work.

HIGH-LEVEL STORSTRØM BRIDGE.

Total length		3200 metres (10,535 feet)
Masnedø approach:	Ten spans of	57·785 metres (189 feet 7 inches)
	Ten spans of	62·230 metres (204 feet 2 inches)
	One span of	57·174 metres (187 feet 7 inches)
Falster approach:	Twelve spans of	57·785 metres (189 feet 7 inches)
	Twelve spans of	62·230 metres (204 feet 2 inches)
	One span of	53·340 metres (175 feet)
	One span of	57·307 metres (188 feet)
Navigation-spans:	Two side spans of	102·3 metres (335 feet 8 inches)
	One centre span of	136·27 metres (447 feet 5 inches)

Approach gradients, 1 in 150
Clear headroom under navigation-span, 26 metres (85 feet 4 inches)
Maximum depth of water, 13·8 metres (45 feet 3 inches)
Maximum height of piers, 38·2 metres (125 feet 4 inches)

Total concrete in piers, etc.	100,000 cubic metres
Total steel construction	21,000 tons
Total steel castings	800 tons
Total steel piling	4,300 tons

LOW-LEVEL MASNEDSUND BRIDGE.

Length		222 metres (728 feet 4 inches)
Fixed spans:	Five spans of	31·5 metres (103 feet 4 inches)
Bascule-span:	One span of	28·4 metres (93 feet 2 inches)
Bascule-span:	Clear opening of	25 metres (82 feet)
Gradient		1 in 333·3
Clear headroom		5 metres (16 feet 5 inches)
Maximum depth of water		11 metres (36 feet 1 inch)
Total concrete in piers		8,000 cubic metres
Total steel construction		1,200 tons

RAILWAY AND ROADWORKS.

Total quantity of earthwork in embankments . .	1,800,8000 cubic metres
Plus sand-material in base of Falster embankment .	270,000 ,, ,,
Mud excavation extra	125,000 ,, ,,
Grassed slopes	335,000 ,, ,,
Stone-pitched slopes	8,000 square metres
Concrete work	4,000 cubic metres
Total railway track laid	15·4 kilometres
Quantity of railway ballast	40,000 cubic metres
Area of roadway laid	90,000 square metres

SPECIAL NOTES IN CONNEXION WITH THE STEELWORK DESIGN.

Choice of Plate-Girder Design.

A few comparisons between the adopted plate-girder design and a conventional deck-type lattice-girder design for the approaches consisting of 70-metres simple spans are given below.

Quantities per foot of bridge.	Plate-girder design.	Lattice-girder design.
Steel in superstructure: tons	2·08	2·03
Concrete in foundations: cubic yards	11·3	9·9
Steel in foundations: tons	0·46	0·36

The average rates quoted for both designs were practically the same. The advantage in cost therefore appears to rest with the lattice-girder.

The plate-girder possesses some advantage in appearance, especially due to the disproportionate depth of the lattice-spans near the end of the bridge where the height above water-level is reduced.

The plate-girder gains over the lattice-girder in freedom from secondary stress, and, for the same permissible stress, probably possesses a higher load-factor.

The shallower depth of the plate-girder for the same stiffness allows a larger headroom below the girders, although this was hardly of primary importance in this case.

The exposed area of the plate-girder requiring painting is only about half that of the lattice-girder, with a corresponding reduction in maintenance costs.

The difference in first cost per foot of bridge at the schedule rates amounted to about £4 12*s*. 0*d*., which at 4 per cent. represents an annual interest of £1,840 available to offset the increased maintenance-cost.

Design of Main Plate-Girders.

Web-Stiffening.

Both vertical and horizontal stiffening was employed with a web-plate of uniform thickness, and the spacing of vertical stiffeners was varied accord-

ing to shear stress. The stability of the web between vertical stiffeners under shear and bending stresses was investigated on Timoshenko's theory, and horizontal stiffening was added where required.

The vertical stiffener-angles were packed out from the web to the thickness of the covers on the central longitudinal web-splice and terminated at the toes of the flange-angles. They were connected to the horizontal legs of the flange-angles by cleats and gusset-plates, thus avoiding fitting and simplifying shop-work.

Flanges.

Wide flanges are necessary in large plate-girders to avoid unduly thick material. Unless large angles are available too much material is concentrated in flange-plates, with consequent difficulty in transmitting longitudinal shear to the web. In this case the use of 12-inch by 12-inch angles produced a reasonably balanced flange and provided adequate width for flange-covers on the underside of the angles.

Design of Navigation-Spans.

The type of design adopted for the three navigation-spans harmonizes well with the outline of the bridge-approaches, and retains the clean appearance of the general plate-girder construction. It also had the great advantage of relative ease of erection as compared with a lattice construction.

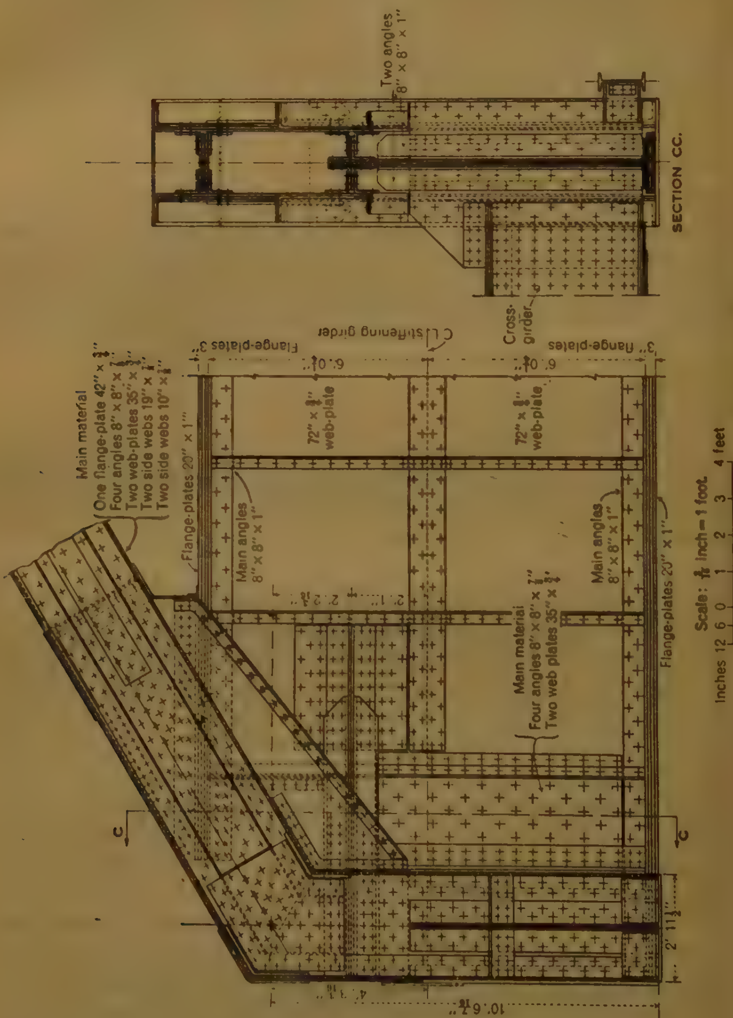
The actual weight of steel in the three navigation-spans as constructed was 3,460 tons, whilst the weight required for a normal through-type cantilever-design for the same openings would have been 3,060 tons—the latter figure being taken from a detailed design. The arch construction thus required 13 per cent. more steel.

The arch was treated as a flexible rib with frictionless joints at the panel-points. It was assumed to be constrained by the hangers to follow the deflexions of the stiffening girder at deck-level, which also acts as the arch-tie. The stiffening girder was assumed to be capable of distributing any loading uniformly to the arch-rib. The connexions of the arch to the stiffening girder were placed above the centre-line of the latter to help to equalize the stresses in its flanges. A calculation of secondary stresses set up in the arch-rib, as a check on the assumption of frictionless joints in the rib, indicated that these were only of the order of 10 per cent. of the primary stresses, thus providing a check on the original assumption. Unless special means were taken to eliminate them, much higher secondary stresses would be anticipated in any usual form of lattice-girder design. The bevels of the hanger-connexions were set out so that their alignment should be correct under full dead-load. The splice-material was designed to transfer the whole load at each joint.

The only detail of construction which presented serious difficulty was

the connexion of the arch-rib to the single-webbed stiffening-girder (*Figs.* 16). The large horizontal component of the arch-rib stress, which amounted to 1,530 tons in the central span, was taken by two horizontal diaphragms,

Figs. 16.



CONNEXION OF ARCH-RIB TO STIFFENING GIRDER.

equally spaced about the springing point. The upper diaphragm connecting direct to the top flange of the stiffening-girder, and the lower to the web, a little above the centre-line. Very careful detailing was necessary to provide access for riveting at this point.

Bridge Bearings.

Contrary to usual English practice, all bearings were of the rocker, as distinct from the knuckle-pin, type, whilst the expansion-bearings of the approach-spans, both on the piers and at the ends of the suspended spans, were of the single rocker instead of the roller form. Extremely massive castings were adopted for all bearings.

The mushroom type of rocker adopted permits angular displacement of the upper and lower elements, both longitudinally and transversely, with very little frictional resistance, although high pressures are developed at the point of contact. The knuckle-pin type of bearing permits only longitudinal rotation, whilst the frictional resistance to this is often high enough to distort the chord-members.

The large single-rocker type of expansion-bearing effects a saving in the height of the pier, is of simple and robust construction, and movement is not prevented by any normal accumulation of dirt on the bearing surface. In this case no protection of the bearings against dust and dirt or corrosion was considered to be necessary.

Use of High-Tensile Steel.

Without the use of high-tensile steel many of the plate-girder details, such as flange-splices, would have proved extremely clumsy, whilst the rivet-grips would have been excessive. High-tensile steel rivets were employed for the connexion of all high-tensile steel parts. Both the structural and rivet-material proved perfectly satisfactory in fabrication, and demanded no modification to the normal shop processes of fabrication.

In general, the whole of the arch-ribs, hangers, stiffening girders and approach main girders, with their stiffeners, were constructed of high-tensile steel. The whole of the deck-material was of manganese steel, with the exception of the end cross girders of the arches, which were of high-tensile steel. The lateral bracing was of manganese or ordinary mild steel, except for the approach-span portal-bracing which was of high-tensile steel. Ordinary mild steel to British Standard Specification was only used for packings and for lightly stressed parts such as handrails, walkways, etc.

The usual rules for the minimum thickness of material were rigidly adhered to, and there was no tendency for the use of high-tensile materials to lead to any flimsiness of construction.

Provision for Maintenance.

Particular attention was paid to the painting of the structure and to provision for maintenance.

The relatively shallow depth of the plate-girders facilitates access from below. The underside of the whole bridge is accessible from painting-travellers, whilst walkways are provided below all arch-ribs.

The total weight of steel in painting-travellers and runways was 630 tons.

Temporary Works and Erection.

Messrs. Dorman, Long and Company, Ltd., were throughout working in very close collaboration with their sub-contractors, the Danish firm of Messrs. Christiani & Nielsen, Ltd., the former dealing mainly with the steel superstructure and the latter with the earthworks, piers, and foundations.

Both firms had evolved new and original schemes for executing their own work, and the form of the permanent structure which they put forward in their offer was adjusted so as to suit the erection schemes which they had in view. The Danish State Railways' engineers, realizing the great advantages both in economy and expedition which the Contractors' methods conferred, left a wide degree of freedom to the firms in the choice of structural details. This somewhat exceptional triangular collaboration worked harmoniously and the bridge was built without any serious difficulties or delays in less than the contract time and at a very moderate cost (approximately £2·75 per square foot of surface).

The great length of the bridge to be constructed and the similarity of its various components, below and above water-level, justified the development of special plant designed for repeated service in the construction of foundations, piers, and steel superstructure.

Foundations.

The foundations of the low-level bridge-piers and abutments and the foundations of a few special piers in the high-level bridge were dealt with by the ordinary method of first forming a steel sheet-pile cofferdam and afterwards excavating and concreting within the cofferdam. The large central piers supporting the navigation-spans which were built in this way call for special mention.

Firstly, a timber platform and surrounding waling was set up just above water-level supported on timber piles driven by floating plant in the open water. On this was mounted a pile-driving plant of the drop-hammer type travelling on rails around the periphery of the platform, but before any steel piles were driven a heavy built-up steel-girder waling was put together on the surface, then suspended from the timber staging and lowered through the water to a point about 18 feet below water-level. This waling was oval in shape corresponding to the shape of the foundation. Guided by the timber waling above, and the steel waling below water the outer ring of steel sheet-piles was then hammered down, adjacent piles being spot-welded along the interlock and driven together in pairs to save time. After the cofferdam was pumped out a heavy reinforced-concrete waling was cast in situ at the bottom. The whole hydrostatic head du

to 35 feet of water plus 10 feet of external clay was therefore eventually borne by the three walings—one timber, one steel, and one concrete. The piles were of the K.111 type rolled in England out of "Chromador" steel, and, considered as vertical beams, they were stressed up to about 10 tons per square inch.

After the concrete waling had set, the excavation carried out by hand and by grab was continued to a depth of about 10 feet below the sea-bed, after which the concrete foundation-slab was deposited by means of a low-level floating concreting outfit.

The lower concrete waling was incorporated as part of the foundation-slab. When the pier-shaft had been built up above sea-level the steel piles were cut off at about the level of sea-bottom by divers working from the outside and using underwater gas-burners supplied through flexible tubing with oxygen and hydrogen from the surface.

In the case of some of the steel-pile cofferdams the bottom was not sufficiently firm and impervious to permit of pumping out, and in these cases both excavation and concreting had to be done underwater.

Altogether there were eight piers in the high-level, and five piers in the low-level, bridge which were surrounded by steel sheet-pile cofferdams.

The remaining piers in the high-level bridge, to the number of forty-one, were dealt with by means of floating steel-cofferdams which were called "units."

These units were of two types: (*a*) "internal units" where the steel-pile foundation-ring was driven outside the shell of the unit, and (*b*) "external units" where the piles were driven inside the shell of the unit. There were two "internal units" used to construct twenty-seven pier-foundations where the ground was sufficiently good to permit work to proceed in the dry, and two "external units" used to construct fourteen pier-foundations where the de-watering of the unit was considered inadvisable. The units served four purposes:—

1. As working platforms for operating plant.
2. As guides for driving the ring of foundation-piles.
3. As cofferdams.
4. As shuttering for the underwater portion of the concrete pier-shafts (internal type only).

The units were oval in plan and had two steel skins with air-compartments between the skins.

Before being floated out a few timber piles were driven down in a ring with their tops carefully levelled at about the level of the sea-bottom. The bottom plate of the unit came to rest upon these piles after it had been floated into position and sunk.

In some cases the steel sheet-piles forming an oval ring which enclosed the foundation-slab of the pier were interlocked and hung round the unit in harbour. In other cases the piles were pitched after the unit had been

placed. In the internal type, the piles were hung round the outside skin and in the external type round the inside skin.

These piles were driven down by means of a McKiernan-Terry hammer until the tops of the piles were level with the bottom strake of the unit. To carry out this driving, it was of course necessary for the hammer to be lowered underwater, the hammers being operated with compressed air instead of steam.

This phase of the operation is illustrated diagrammatically in Figs. 17, Plate 1. There was an oval steel waling round the outside of the lower strake of the internal unit and hardwood blocks were fitted in the corrugations of the heads of the piles, so that when the piles had been driven down, the piles with their hardwood-block fillers made continuous near contact with the steel waling. The narrow space between the piles and the waling was then made watertight by divers going down and plugging it by inserting a rope impregnated with tallow. The subsequent pumping out of the water enclosed within the unit caused the rope to be sucked in tightly and to seal the cofferdam (Fig. 18, Plate 1).

In the case of external units there was a V-shaped space between the tops of the piles and the inside sloping shell of the unit, which was sealed with concrete placed underwater.

After the internal unit had been pumped out, excavation proceeded by hand and by grab (Fig. 19, Plate 1), and the concrete foundation-slab was deposited in the dry. In the case of the external units, however, pumping was not resorted to until after the excavation had been completed and the concrete foundation-slab deposited underwater.

The building up of the lower part of the pier-shaft was then carried out, the interior sloping skin of the internal unit itself acting as the form against which to run the concrete (Fig. 20, Plate 1). The concrete having been finished off and carefully levelled at 2·5 metres (8 feet 3 inches) below water, the unit was floated off. Vertical tubes had been built at intervals between the two skins of the unit through which posts could be inserted, and resting on the base-concrete could be jacked down, and so provide necessary uplift to break the bond between the concrete and the internal skin or shutter face of the unit.

In most cases the units lifted quite easily, when the water inside its compartments was pumped out, without jumping up with too great a jerk, but the jacking-posts were a reasonable precautionary measure. Since the units when fully pumped out still drew more than 2·5 metres (8 feet 3 inches) of water, they could not be floated over the concrete pier on a level keel, and it was therefore necessary to water-ballast them on one side and to careen them over the piers (Fig. 21, Plate 1).

It had originally been intended to construct all the piers by means of internal units because it was supposed that the clay floor of the sea was everywhere sufficiently stanch to permit pumping out, but it was discovered, shortly after the commencement of the contract, that these condi-

tions did not always apply, and the "external unit" was devised to provide a safer method on a bad or doubtful bottom.

The underwater concreting was carried out by running fairly plastic concrete down through a 10-inch diameter tremie-pipe so that the fresh concrete was fed in at or near the bottom of the mass all the time and did not therefore come in contact with the water. To commence with, the tremie-pipe was lowered into the compartment to be concreted with a flat steel plate at its bottom end. Concrete was then fed into the top of the tremie-pipe through a hopper above water, the first batch of concrete being protected by means of a jute plug which it pushed down the tube in front of it. It was, of course, impossible to prevent the first few batches of concrete from being scoured or washed rather badly on emerging and spreading out sideways round the bottom of the tube, but afterwards if the tube was carefully manipulated by raising and lowering so as to keep the flow of concrete proceeding steadily it was possible to deposit the concrete with very little loss of cement.

In order that the concrete shall be brought up in one compact plastic mass the volume to be deposited at one time must bear a certain relation to the output capacity of the mixing plant, and in the case of these foundation slabs it was therefore found necessary to subdivide the area into ten compartments by means of vertical shutters lowered through the water and fixed in position by divers. The individual compartments had a superficial area of about 40 square yards, and, the depth of concrete deposited being about 10 feet, the volume worked out at about 120 cubic yards. The tremie-pipe was lowered in the centre of the compartment so that the concrete had at most only about 10 feet to spread out laterally. The surface of the finished concrete sloped from the centre outwards at a gradient of about 1 in 10. Even on the top surface of the concrete there was very little laitance or washing out of the aggregate. Tests made by fixing sloping wooden drums in some of the compartments and afterwards inspecting the concrete which had been formed round about these awkward obstacles showed that the quality of the work was satisfactory. The mix was 1 : $2\frac{1}{2}$: $2\frac{1}{2}$, and of course the rather high percentage of sand and of water in the mix detracted from the strength. The concrete was, however, amply strong for all practical purposes in such a position, and was quite impervious.

While concreting was in progress a duplicate mixing outfit was provided so that a mechanical breakdown of one outfit should not cause a break in the continuity of the operation.

Lower Part of Pier-Shafts.

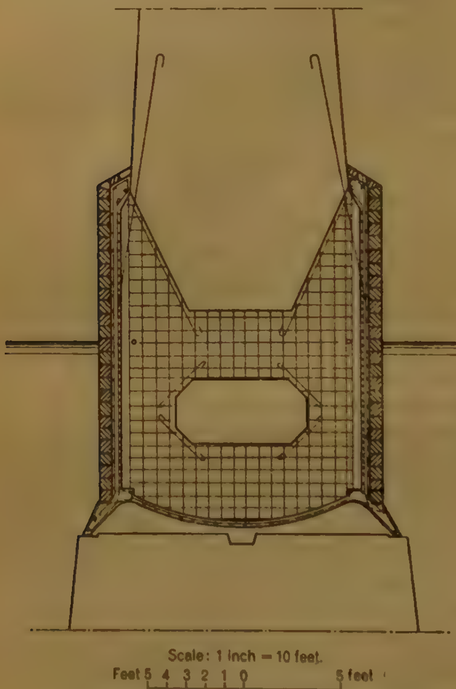
Where the pier-foundations had been built inside interior-units, the lower part of the pier-shafts were formed by running the concrete up against the well-greased internal metal skin of the unit. The base of the shaft formed in this way resembled a prism bounded by a number of plane faces.

The concrete was deposited in the dry from a ship fitted up for the purpose. The top of the prism was carefully levelled off, and was surmounted by a diamond-shaped beading of asphalt mixed with sand about 1½ inch wide by 1½ inch high intended to engage and form a watertight joint with the bottom edge of the "middle-body" described in the next section.

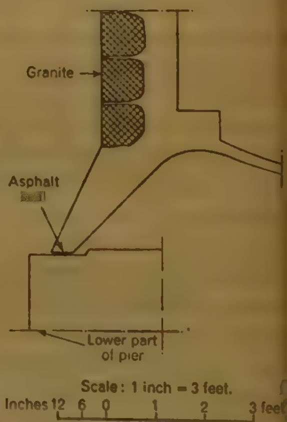
"Middle-Body" of Piers.

The "middle-body" portion of the pier-shaft was built on a slipway on shore. For the sake of lightness it was built hollow with reinforced-

Figs. 22.



CROSS SECTION OF MIDDLE BODY AFTER ERECTION.



DETAIL OF MIDDLE BODY SEATING.

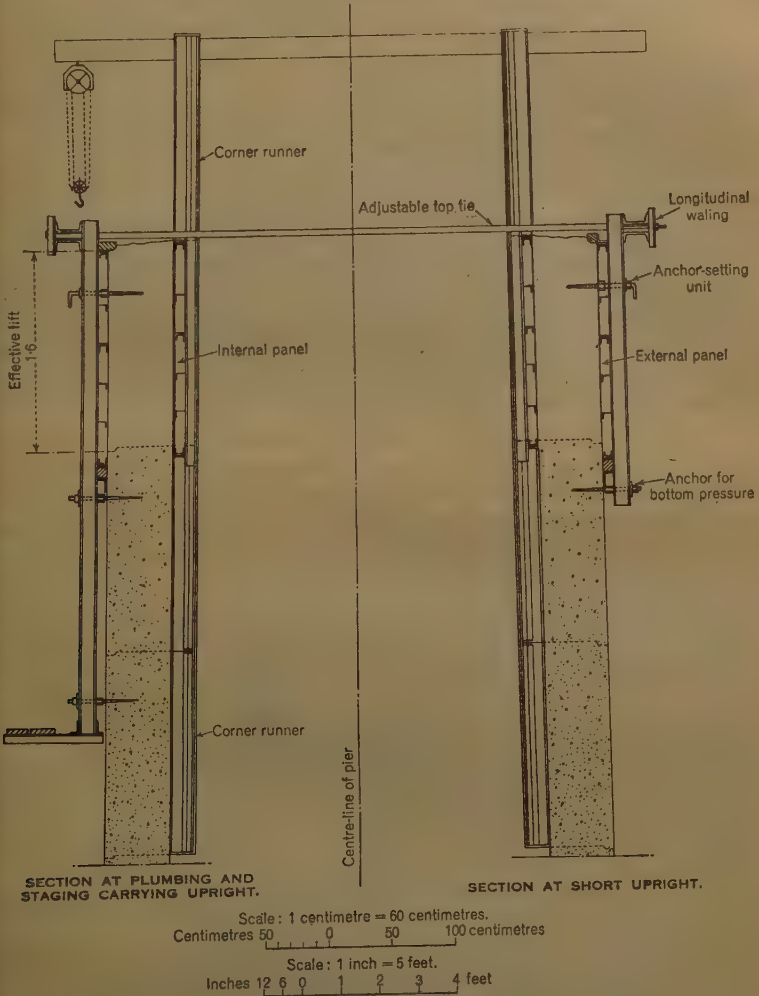
concrete walls. It was faced throughout with granite stones quarried in the Danish island of Bornholm, averaging 8 inches in thickness and laid in courses 8 inches deep.

By doing this work as a repetition-job in a building yard ashore it was possible to ensure the highest standard of workmanship, and the cost was low.

The middle-body part of a pier usually took about a fortnight to construct on the slipway, and immediately one was finished it was lowered

down the slipway where it was straddled by the pair of specially rigged barges and its weight picked up on a series of lifting screws suspended from overhead girders. The middle-body was afterwards floated out

Fig. 23.



DETAIL OF STEEL SHUTTERS FOR CONCRETE PIERS.

between the barges and lowered upon its prepared seat so that the lower bearing edge of the middle-body rested upon the aforementioned asphalt bed and automatically made a watertight joint with the concrete beneath (*Figs.* 22). Two pins projecting below the middle-body engaged with two

funnel-shaped holes left in the top of the base beneath so that the middle-body was bound to lower down into its exact position on the base.

After being lowered the water was pumped out of the middle-body, light reinforcing rods left in the base were then bent up and the interior space filled with concrete. The middle-body thus became one monolithic structure with the base.

Upper Part of Pier-Shafts.

The upper part of the shafts projecting above water were shuttered with steel shutters raised in lifts of approximately 5 feet at a time.

The shutters stretched between a series of steel soldiers bolted to the concrete and raised by means of chain blocks from a central steel gallows structure which was itself raised in stages as the work climbed upwards. The whole was specially designed for Messrs. Christiani & Nielsen, Ltd., by Mr. C. Parry, of London. Details of the shutters are shown in *Fig. 23.*

The concrete in the shafts was gauged, mixed, hoisted, and deposited through shoots from off a specially equipped ship (*Fig. 24*). There were two such ships in use, one for the low-level concrete work on the foundations and lower shafts and the other for the high-level work.

To do the work in the time, operations had to be organized so that each of the four units and each of two concreting outfits, the middle-body building slipway, the floating pile-drivers, and also the shaft-shuttering and other plant, were continuously in use without idle periods, and at the same time a steady sequence of completed piers had to be provided to absorb the output of the steel bridge-span construction yard. As there were a great many different types of piers and spans, the task of co-ordinating progress was a matter of complexity and had to be the subject of continuous forward-planning and arranging as the work proceeded.

Building of Steel Spans.

All the bridge-spans for both of the bridges were built on a single slipway constructed for that purpose on the south shore of the island of Masnedø, near where the construction yards for general works purposes were situated.

The building berth was at ground-level and was spanned by a goliath-crane running on a track parallel to the shore.

Before leaving England the girders were riveted up in as large pieces as possible consistent with a limit of 25 tons in weight and not being too large to fit into the holds of the carrier-ships. On arrival the ships were berthed at a reinforced-concrete deep-water jetty specially constructed at Masnedø, and were discharged by means of a 25-ton derrick-crane on to rail-bogies, which were run direct from the jetty to be unloaded and stacked by the goliath-crane about 300 yards distant. Smaller pieces of steel were discharged by 5-ton travelling cranes on storage areas apart from the main

Fig. 24.

FLOATING CONCRETING PLANT.

Fig. 26.

FLOATING CRANE CARRYING A HALF NAVIGATION-SPAN.

5197.

Fig. 27.

NAVIGATION-SPANS UNDER CONSTRUCTION.

Fig. 28.

ERECTION OF END SPAN.

storage ground beneath the goliath-crane. It was in this way possible to store a large quantity of steelwork on the site, so providing elasticity between the shop-fabrication work in England and the span-erection programme in Denmark.

Whenever a bridge-span was built complete with its rocker-bearings on the building-berth, it was jacked up and lowered upon special carriages running upon steel ball-race tracks laid on a pair of low concrete runways. The runways extended from the building berth on shore for a distance of 190 feet out into the channel dredged for the reception of the floating crane. The four carriages, each carried by thirty steel balls 3 inches in diameter, supported the weight of the bridge-span, amounting to nearly 500 tons in some cases. The concrete runways were supported on timber-piled foundations. The whole arrangement worked admirably except for a slight difficulty in getting suitable hardened steel strips to carry the balls. Each ball was supposed to bear a load of up to 4 tons, but owing to small irregularities in the tracks and other factors was liable to carry a good deal more momentarily, and the point-pressure on the base-track was therefore rather high. It was found that specially hardened steel strips tended to crack and spall under the balls, whereas ordinary mild-steel strips tended to form grooves in which the balls ran.

Erection of Steel Spans.

The spans for the low-level bridge were put together in the building berth on timber packings of about the same height as the bridge-piers.

The spans were then run out on the runways and picked up on a timber platform built on top of and supported by two steel barges beneath.

Since the spans of the Masnedsund bridge were too short to span the yard-runways, temporary braced-steel extension-pieces were provided, connected to each end of the span, upon which they travelled.

The picking-up process was effected by pumping out the barges which had been previously half-filled with water.

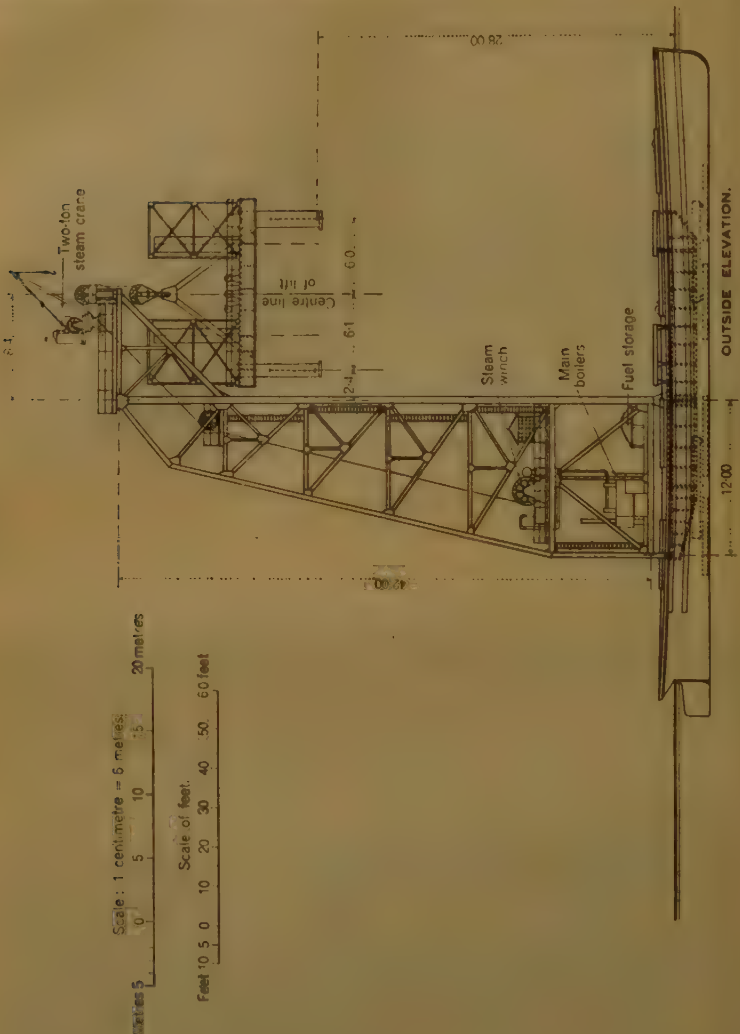
The two barges used for this purpose were provided with a series of replaceable timber superstructures according to the purpose for which they were required.

The two barges when rigged in one such guise were used for floating out the low-level bridge-spans above mentioned; in another guise they carried a long crane-jib and gear which was used for erecting the cast-steel bridge-bearings on top of the completed piers; in a third guise they carried a tower for supporting the end of one of the shore-spans of the main bridge when "rolling them in" as will be described later; and in another guise they acted as a platform for bearing the sand-blasting outfit, and as a floating workshop for men engaged in building stagings.

In a final guise the two barges were taken apart and used to support the two oversailing members of the big portable timber trestle.

In order to float out the spans of the main bridge, weighing up to 500 tons apiece, to hoist them to a height of 90 feet above water-level, and to place them upon their bearings, a floating crane, details of which are

Fig. 25 (a).



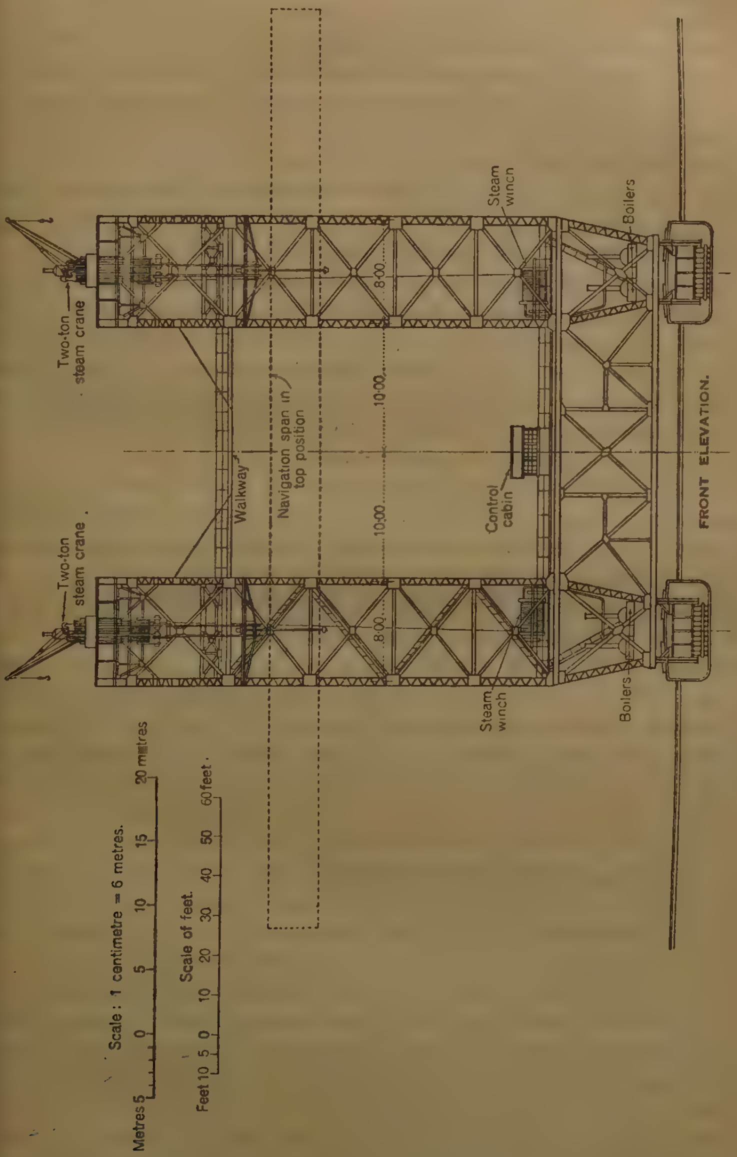
FLOATING CRANE: OUTSIDE ELEVATION.

given in *Figs. 25* (*a*) and *25* (*b*), was eventually designed and constructed. This consisted of three parts :—

(*a*) the floats.
(*b*) the steel skeleton.
(*c*) the lifting and mooring mechanism.

As regards (*a*), two steel sea-going dumb barges of 750 tons deadweight which had been in use for about 30 years as grain carriers on the river

Fig. 25 (b).



FLOATING CRANE: FRONT ELEVATION.

Elbe were purchased secondhand in Germany, towed through the Kiel canal and delivered in Copenhagen. These old barges were in a perfectly

sound condition and were obtainable at a fraction of the cost at which ne barges or new pontoons could have been built.

As regards (*b*), the structural steelwork consisted of the two towe approximately 140 feet high joined together near the bottom by a b lattice box-girder. Each tower had, at its lower extremity, a kind of fo consisting of a pair of cross-connected plate-girders which were designed spread the weight of the towers and load evenly on the bottoms of th barges.

The steel skeleton, made up of the two towers, cross girder, and foo girders was a single rigid unit. In order to get the foot-girders into th barges the bulkheads had to be cut through, and holes had also to be cu through the barge-decks to pass the legs of the towers.

The weight of the steel skeleton and of the bridge-span carried by th skeleton was transmitted through the foot-girders to a grillage of pla steel joists beneath, at two distributing or nodal points in the length of th barge, one such point being forward and the other aft of the amidsh section. There was no rigid connexion between the hull of the barge an the steel skeleton, and the distribution of load along the barge-bottom every condition of service was fairly even.

When there was no span aboard, the weight of the skeleton put th barges a little down by the stern, but when the span of greatest weight wa aboard or being lifted the barges were on a level keel, although not eve then were they loaded down to capacity.

Projecting upwards through the deck of the barges from the forwa part of the toe-girders were rigid steel posts and platform-member designed to support the steel bridge-span when at rest in the lowere position before being lifted. The supporting platform was at a slightl higher level than the barge-decks and was part of the steel-skeleton syste and therefore not connected to the barge-decks, so that the dead loa whether at rest or uplifted could not subject the hulls to any excessiv strains.

The two barge-hulls were connected near their bows by a steel tie-gird above water-level, designed to withstand any relative distortion. Th girder was designed to be as flexible as possible consistent with strengt and its connexion to the hulls was distributed over as great an area a possible to reduce local concentration of stress. Apart from the stee superstructure, it formed the only connexion between the two barges.

Provided that the barge did not have to be navigated in waves great than 5 feet from crest to trough, there were no serious stresses to b apprehended as a result of diagonal hogging or sagging. When operatin in the Storstrøm the craft was not taken out of dock when the wave were as large as this. The most anxious time was when the barge were being towed from Copenhagen to the site by way of the Soun and the open Baltic, but this passage was carefully timed and successfull negotiated.

The mechanical equipment comprised both lifting and mooring mechanism. The lifting mechanism was operated by two steam-driven 25-ton worm-drive main winding winches manufactured in England by Messrs. Stothert and Pitt.

In addition to the main lifting winches there were six steam-winches, most of which were mounted beneath the barge hatchways so as to be out of the way, each of which hauled on a wire-rope mooring, there being one fore and one aft, and one side or breast mooring-rope on each side of the outfit. These winches were used to warp the craft out of and into the dock-channel and to make fast to the bridge-piers on either side when fixing a span.

The rough adjustment having been made by means of the deck-winches, the delicate final movements required to bring the span exactly into position over its bearings were effected by means of hand crab-winches fixed on the lifting cradles. There were six of these winches, three operating at one end of the span and three at the other. One of these ropes gave a direct longitudinal pull between the suspended span and a ring-bolt fixed on the top of the pier, the other two gave side pull in either direction and were fastened to the same ring-bolt.

When lowering a span into position it was necessary to control simultaneously the six deck-winches, the six cradle-winches, and the two winding winches. Control was in the hands of a central operator on the floating crane who was connected by telephone to an engineer on each pier and to a foreman standing aloft on the span between the cradles who passed orders on to the men on the hand-winches. The central operator, being on a staging just above the barges, could pass direct orders by hand-signal to the men on the deck-winches and also to the main winding winches. The engineers on the piers kept the central operator advised as to the exact movement of the span with regard to the two bearings each on his particular pier. The fixed-bearing end of a girder was usually first contacted and the rocker-bearing immediately afterwards.

Care had also to be taken not to venture out with the floating crane on windy days, because with a wind exceeding 20 miles per hour in velocity the wind-pressure on the large exposed area of the crane-towers and of the span carried by the crane was such as to render the craft uncontrollable by the three tugs in attendance.

The total time taken for the erection of the first span to be placed was 12 hours, although later, when practice had made the crews more adept, the time taken in warping out of dock, hoisting the span, towing to the site, setting the span, casting off and returning to dock only occupied 4 to 5 hours.

Two anchor spans had to be erected before the suspended span between them could be put up, but the operation of floating out, hoisting, and placing the spans was very similar in the two cases.

The erection of the three navigation-spans had to be carried out by a

more complicated method, as the central navigation span weighed 1,520 tons, or approximately three times the weight which the floating crane could lift. To overcome this difficulty the chord or tie-girder portion was built in two halves in the building yard and first one half and then the other half was erected by the floating crane. This necessitated providing a temporary support or pier in the middle of the span, and as after the two halves had been erected and riveted together they were not strong enough to support their own weight the temporary support had to be left in position until the arch had been built.

When the first half of the first navigation-span to be erected was ready for floating out, a 20-ton crane was built upon it and floated out together with the span (*Fig. 26*, facing p. 416).

After the other half-span had been erected and the two were spliced together, the derrick-crane (moving on a rail-track on the deck of the span) assembled the arch-rib members as it rolled forward. This work was commenced at one end of the arch-rib under construction and was carried right through until a closure was affected on to the chord member at the far end. This procedure was adopted as being preferable to commencing the erection at each end of a span and closing the arch-rib in the centre (*Fig. 27*, facing p. 417).

The temporary support-pedestal under the middle of the chord-member was capable of being raised or lowered by means of hydraulic jacks. In order to splice the ends of the opposing half-chord members the centre support had to be lowered about 18 inches below the normal position. After the splice was made the pedestals were jacked up to a little above normal while the arch-rib was building. The jacks were finally used when the time came to close up and rivet the last joint between the arch-rib and the chord-member. When the span was completely built the 20-ton crane was left standing just clear of the end of the span ready to be run forward on to the chord of the adjacent navigation-span.

The temporary pier-support consisted of two parts, the base and the tower. The base was a heavy piled timber platform, constructed of spruce, whilst the tower was built of spruce capped with oak, the posts being formed of groups of roughly squared 10-inch by 10-inch logs and the bracings consisting of groups of round logs or half-round logs with the bark on.

The cost of building the tower, which weighed 120 tons, was about £1,000, and it was used for the erection of each of the three navigation-spans.

Modified constructional methods were used for erecting the spans adjoining the shore at both ends of the bridge as the shallowness of the water made it impossible for the floating crane to be used.

The principle adopted was to erect the shore-span temporarily in the second span out from the shore and afterwards to traverse it horizontally over into position. For this purpose it was in one case necessary to build on a part of a third span with a temporary rigid connexion when making

up the shore-span so as to get the necessary length for the traversing operation.

The operation of traversing the shore-span at the south shore-abutment is shown in *Fig. 28* (facing p. 417), where it will be noticed that the rear part of the span was supported by a tower carried on the auxiliary floating barge-outfit previously mentioned, and that a temporary trestle carrying rollers had to be erected at the edge of the water to carry the forward end of the moving span as it approached the abutment.

The dovetailing of the steel erection to fit in with the programme of pier-construction was a matter of very careful planning in advance. It was facilitated by the provision of storage-room for five complete bridge-spans to stand on the launching slipway, whereby the work of building spans was enabled to proceed steadily at a speed of between two and three complete spans every month from shortly after the start until the finish.

The bascule-opening span of the Masnedsund bridge was the only complete span to be built piecemeal. The portion comprising the trunnions and counterweight was first built in the horizontal position on its bearings in the hollow pier and then rotated so that the counterweight portion was lowered into the chamber. The leaf of the span was next erected in the upright position, this being necessary in order to leave the opening clear for the passage of ships. The compartments in the counterweight portion were at the same time loaded with old flat-bottomed rails grouted into a solid mass.

Cleaning and Painting.

After the steel was delivered at the site it was exposed to the weather in the unpainted state before and during assembly, and after erection for at least 12 months, to get rid of the mill-scale.

The whole of the exposed surface was then cleaned by sand-blasting in situ until the metal was perfectly clean and burnished, whereupon it was sprayed with three coats of paint. A final fourth coat of aluminium paint was applied with a brush.

The arch-rib and hanger-members of the navigation-span girders were sand-blasted piecemeal in a shed built for that purpose, and the first two coats of paint were sprayed on immediately after sand-blasting.

It was found that the sand-blast gave a perfectly clean burnished metal-surface upon well-rusted steel surfaces, but was ineffective in removing mill-scale when unloosened by rust. The extra cost of sand-blasting over and above ordinary scraping and cleaning was about 3*s*. per ton.

ACKNOWLEDGEMENTS.

The Authors occupied the following positions in connexion with the construction of the bridge: Mr. G. A. Maunsell, B.Sc. (Eng.), M. Inst. C.E.,

sometime Managing Director "Dansk Engelsk Staalkonstruktion Aktiese-skab" (the Danish company formed for the purpose of building the bridge and Mr. J. F. Pain, M.C., B.Sc. (Eng.), Assoc. M. Inst. C.E., Chief Enginee Messrs. Dorman, Long and Company, Ltd. (Bridge Department).

The works which form the subject of this Paper were built under th supervision of the Chief Engineer, Mr. H. Flensborg, aided by his colleagu in the service of the Danish State Railways, and by the State Railwa Bridge Consultant, Professor Anker Engelund.

The successful achievement of the project both in its inception and i execution was brought about from start to finish very largely by the effor and special skill of Messrs. Christiani & Nielsen, Copenhagen.

The Paper is accompanied by nine sheets of drawings and fourtee photographs, from some of which Plate 1, the Figures in the tex and the half-tone page-plate have been prepared, and by the followi Appendix.

APPENDIX.

The Steel Bridge Designs.

Materials.

The specification for the steels to be used in the bridge arrived at after discussion between the Danish State Railways and the British manufacturers was as follows :—

Table I.—Chemical and Physical Requirements of Steel.

	High-tensile steel.	Mild steel.	High-tensile steel rivet bars.	Mild-steel rivet bars.
Percentage of—				
Phosphorus	About 0·05	Less than 0·5	About 0·05	Less than 0·045
Sulphur	About 0·05	Less than 0·5	About 0·05	Less than 0·045
Manganese	About 0·7–1·0	Not specified	Not specified	Not specified
Silicon	About 0·2			
Carbon	Less than or about 0·3			
Chromium	About 0·7–1·1			
Copper	About 0·25–0·5			
Ultimate tensile strength. Max.	43·1	37	37	
In tons per sq. in. Min.	36·8	32	31	
Min. yield point (thickness ≦1¼″)	22·8	18·4	—	—
(In tons per sq. in.) (thick- >1¼″)	20·3			
Min. elongation (per cent.) (thickness ≧⅜″)	17	20	20	According to B.S.S. No. 15
(thickness <⅜″)	15			
Cold bend of 180 degrees around pin with diameter :—				
Specimen cut in direction of rolling	2 T (thickness)	1·5 T	—	
Specimen cut across direction of rolling	3 T (thickness).			

The materials adopted to satisfy this specification are briefly described on p. 426.

High-Tensile Steel.

High-tensile parts were made from Messrs. Dorman, Long's "Chromador" stee a chrome-copper alloy steel conforming to the requirements of B.S.S. 548. Thi material possesses good rust-resisting properties, whilst the segregation of the chromiu or chromium carbide is reduced to a minimum provided that the carbon content is kep below 0·26 per cent., as indicated by the fact that in the great majority of cases th bend-test specimens were closed flat without fracture. In cases where the carbon percentage exceeded 0·26 normalizing was resorted to.

As a matter of interest all bend-test specimens were closed flat after the specifie test, and no difficulty was found in satisfying this condition.

Mild Steel.

The specified yield-point of 18·4 tons per square inch could not be obtained wit ordinary mild steel to British Standard limits. A special material was therefor adopted with a manganese percentage of up to 1·0–1·2 per cent. The tensil strength was raised to from 32 to 37 tons per square inch.

Lightly stressed parts were made from ordinary mild steel.

High-Tensile Rivets.

High-tensile rivets were used with the high-tensile material. The material adopted after tests was of generally similar composition to mild steel but with the addition of 1 per cent. chromium. It proved entirely satisfactory. A shear strength of no less than 27·5 tons per square inch after driving was aimed at. It was found that with material having an initial tensile strength above 36 tons per square inch the rivet heads hardened unduly. The slip under load of joints made with high-tensile and mild steel rivets was investigated. The clamping action of the high-tensile rivets i relatively less efficient than with mild-steel rivets.

Mild-Steel Rivets.

Material complying in all respects with B.S.S. No. 15 was adopted.

Normalizing.

The grain-size of material 1 inch and over in thickness becomes appreciably larger than that of thinner sections, due to the smaller amount of work to which it can be subjected in normal rolling practice. As a precaution, therefore, all material over 1 inch in thickness was normalized by heating uniformly above 750° C. (but not ex- ceeding 800° C.) for a period depending on the thickness of the material, and subsequently cooling in air after withdrawal from the furnace. This treatment slightly lowered the tensile strength and raised the elongation from an average of 18 per cent. to an average of 22 per cent.

Permissible Stresses.

The specified permissible stresses expressed in tons per square inch are summarized below :—

Tension and Bending.

Combination.	Loading.	Mild steel.	High-tensile steel.
(1)	Dead load + live load + dynamic action	9·85	12·7
(2)	1 + wind + train-thrust + temperature + etc.	11·75	14·62
(3)	During erection: dead load + live load.	11·75	14·62
(4)	(3) + wind.	12·70	15·85
(5)	Lateral and sway-bracing and braking-girder. Wind or temperature or train-thrust.	8·88	11·12
(6)	In laterals, etc.: Wind + train-thrust + temperature + etc. or during erection.	11·12	14·29

Shear in girder-webs or on rivets = 0·8 × (tensile stress above).
Bearing on rivet-holes = 2 × (tensile stress above).

Permissible compressive stresses are based on Ostenfeld's formula and the Euler curve, allowance being made for the percentage of rivet holes deducted from the section (minimum 12 per cent.).

Ostenfeld's formula may be expressed as :—

$$f = \frac{\Sigma}{n}\left(1 - \frac{\Sigma}{532{,}000}\frac{l^2}{k^2}\right), \text{ or } f = \frac{\Sigma}{n}\left(1 - \frac{x-12}{100} - \frac{\Sigma}{532{,}000}\frac{l^2}{k^2}\right),$$

where

f denotes the permissible average stress-intensity,
Σ ,, material-coefficient corresponding approximately to yield-stress in simple compression = yield-stress in tension + 0·3 (ultimate stress in tension — yield-stress in tension).
(For high-tensile steel $\Sigma = 27{\cdot}2$ tons per square inch.)
(For mild steel $\Sigma = 21{\cdot}2$ tons per square inch.)

n denotes the load-factor, chosen as follows :—

Load-combination	(1)	(2)	(3)	(4)	(5)	(6)
n	2·8	2·4	2·4	2·2	3·2	2·5

l denotes the unsupported length of column,
k ,, radius of gyration in direction under consideration,
and x ,, percentage deduction from area for rivet-holes (not less than 12 per cent.).

The Euler formula can be expressed as :—

$$f = \frac{132{\cdot}5}{n} \times \frac{1000}{\left(\frac{l}{k}\right)^2}, \quad \text{or } f = \frac{132{\cdot}5}{n} \times \frac{1000}{\left(\frac{l}{k}\right)^2} \times \left(1 - \frac{x-12}{100}\right).$$

Euler's formula is to be used when :—

$$\frac{l}{k} \begin{smallmatrix}=\\>\end{smallmatrix} \sqrt{\frac{112 - x}{100}} \times 1{\cdot}235 \quad \text{for mild steel.}$$

and

$$\frac{l}{k} \begin{smallmatrix}=\\>\end{smallmatrix} \sqrt{\frac{112 - x}{100}} \times 0{\cdot}965 \quad \text{for high-tensile steel.}$$

For latticed columns, the unsupported length l is to be multiplied by a factor

$$y = 1{\cdot}1\sqrt{1 + \left(\frac{2^T}{E} \times \frac{\sec a}{\sin^2 a} \times \frac{F_f}{F_d}\right)},$$

where T denotes the ultimate stress for a solid column of some value of $\frac{l}{k}$,

	E	,,	value of Young's modulus,
	a	,,	angle between lacing-bar and chord,
	F_f	,,	gross area of the flange,
and	F_d	,,	gross area of lacing cut by any one cross section.

Star-angle sections battened together with batten plates, each having two rivets line at not more than nine times the width of the smallest angle apart, can be treat as solid columns.

For battened columns, the unsupported length l is to be multiplied by a factor

$$y = 1{\cdot}1\sqrt{I_t + 0{\cdot}5 \times \frac{I}{I_f} \times \left(\frac{c}{l}\right)^2 + T \times \frac{F_f \times C \times h}{5 \times E \times I_t}},$$

where I denotes the gross moment of inertia of the whole column,

	I_f	,,	gross moment of inertia of part of the column between batten about an axis at right angles to the plane of the battens,
	I_t	,,	gross moment of inertia of the battens about axis at righ angles to the plane of the battens,
	C	,,	distance between the batten-plates,
and	h	,,	distance between the flanges connected by the battens.

The ratio $\frac{c}{k}$ for the part of the column between battens must not exceed $\frac{l}{k}$ for th whole column, nor must it be greater than 40.

Bearings.

For cast-steel bearings, the permissible stress for pressure and bending is

8·9 tons per square inch for combination (1).
10·2 ,, ,, ,, ,, for combination (2).

Permissible shear-stress = 0·8 × stress for pressure and bending.

Permissible bearing-pressure p for bearings with line or point contacts whe unloaded :—

Bearings with :	Combination (1), p : tons per square inch.	Combination (2), p : tons per square inch.
One or two rollers	5·1	5·7
Three or more rollers	4·45	5·1
Point-contact	7·0	8·25

$$\text{For roller-bearings } \frac{1}{R} = \frac{5{\cdot}5p^2L}{EP},$$

where R denotes the radius of the roller in inches,

p	,,	permissible bearing-pressure in tons per square inch,
L	,,	length of contact in inches,
E	,,	coefficient of elasticity = 14,000 tons per square inch,
and P	,,	total load on the roller in tons.

For point-contacts (contact between two spherical surfaces),

$$\frac{1}{R_1} + \frac{1}{R_2} = \frac{4}{E} \times \frac{p^{\frac{3}{2}}}{p^{\frac{1}{2}}},$$

where R_1 and R_2 denote the radii of spheres in inches (+ for convex, − for concave surfaces).

For "Haberkalt" rocker-bearings, the following equation must be satisfied for stability:

$$h = \frac{\overline{r_1}}{\dfrac{1 - r_1}{R_1}} + \frac{\overline{r_2}}{\dfrac{1 - r_2}{R_2}},$$

where h denotes the height of the rocker,

r_1	,,	radius of the top casting,
R_1	,,	radius of the upper end of the rocker,
r_2	,,	radius of the lower casting,
and R_2	,,	radius of the lower end of the rocker.

(All the above dimensions are in inches.)

Design of Stiffeners.

Stiffeners are designed to take a direct load as struts with unsupported length $l = 0{\cdot}85$ (depth of girder).

Intermediate stiffeners are designed to take a load

$$= \frac{\text{Shear}}{4} \times \frac{d}{\text{girder-depth}},$$

where d denotes the sum of distances of adjacent stiffeners.

Stiffeners at cross-girders are designed to take the maximum cross-girder load in addition to the above load.

Discussion.

The Authors showed a number of lantern-slides illustrating the work described in the Paper, some of which were shown in *Figs. 29* to *33*. *Fig. 29* showed in the foreground one of the floating cofferdams placed in position between the rows of temporary dolphins, and in the distance the Masnedø abutment. *Fig. 30* showed the bascule-span under construction in the raised position, that position being necessary in order to permit ships to pass. In the foreground of that view was the temporary service bridge, with its primitive bascule opening span in the lowered position. *Fig. 31* (facing p. 431) showed one of the navigation-span piers with its surrounding steel-pile cofferdams, and with its suit of steel climbing shutters fixed ready for concreting. *Fig. 32* (facing p. 431) showed one of the low-level bridge-spans being floated out from the building-yard. The curious steel projections at both ends of the span were temporary fixtures to give additional length when the spans were rolling out along the slipways. Those slipways, having been designed for the spans of the main bridge, were too wide apart for the low-level bridge-spans to stretch across. *Figs. 33* (p. 431) gave typical sections of some of the piers in foundations used in the low-level bridge over the Masnedsund. The section of the bascule pier showed the chamber in which the weighted hinder end of the bascule sank down when the leaf was opened.

Mr. Ralph Freeman did not propose to refer to the foundations or to the piers, except to point out that when Professor Anker Engelund described the Little Belt bridge he said that he had estimated that pier-settlements of the order of 1 foot would take place over a long period. It would be of interest to know whether any corresponding settlements were expected in the piers of the Storstrøm bridge; had any settlement been observed, and if so, was it greater in the case of the piers in which the concrete had been deposited through the water?

The characteristic feature of the superstructure was the use of somewhat massive forms of plate-girder construction instead of the more conventional form of lattice-girder. He thought that the use of lattice-girders for those spans would have been appreciably less costly and would have involved a smaller area for painting than the plate-girders actually used. On p. 406 a comparison was made in the case of the approach-spans, but it seemed to him that the figures given required some explanation. The comparison was between the girders of the bridge (which averaged

Fig. 29.

UNIT IN POSITION.

Fig. 30.

MASNEDSUND BRIDGE: ERECTION OF BASCULE-SPAN.

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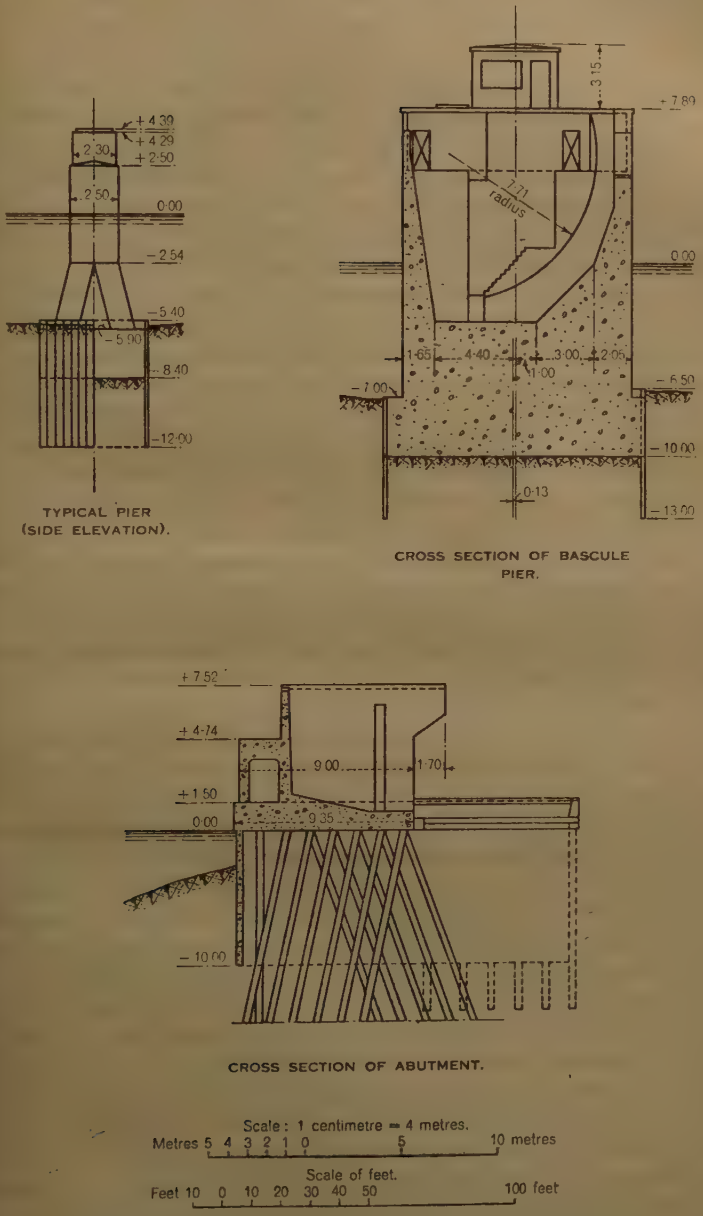
Fig. 31.

STORSTRØM BRIDGE: SHAFT OF A NAVIGATION-SPAN PIER.

Fig. 32.

MASNEDSUND BRIDGE: FLOATING OUT A SPAN.

Figs. 33.



MASNEDSUND BRIDGE: PIER- AND ABUTMENT-DETAILS.

approximately 60 metres span) and a 70-metre simple span. It showe
that the difference in cost was £4 12*s*. per foot. The cost of the bridg
worked out at about £120 per foot; presumably the steel-work an
foundations must have cost between £100 and £110, so that the savin
was approximately 4½ per cent. He found it difficult to reconcile th
with the quantities given; they seemed to represent something lik
6 or 7 per cent. It had to be borne in mind, also, that the compariso
was between 70-metre spans on piers and a bridge built with 60-met
spans; if 60-metre spans had been used, the weight of steelwork would hav
been reduced to about 1·7 ton per foot, representing a reduction of nearly
per cent., and in that case the saving by using lattice-girders on approx
mately 60-metre spans would have been fully 10 per cent. The point wa
important, because the Storstrøm bridge was a very large structure an
embodied an unusual form of girder for the approach-spans, but he doubte
very much whether it was really justified on grounds of economy. H
also questioned whether the estimated area of painting, which was give
as 100 per cent. excess in the case of lattice-girders, was really correct
he thought that if the area had been calculated for 60-metre spans instea
of for 70-metre spans the excess would have been more nearly 50 pe
cent., and much less if members of box form had been used. It wa
of interest to call attention to the arrangement of the approach-span
where alternate spans were extended as suspended spans. It was
method of construction which would effect an economy in bridges wit
multiple spans, and it was surprising that it was not more frequentl
adopted. The arrangement apparently led to no special difficulty
construction or erection in the Storstrøm bridge.

He hoped that the Authors would find it possible to give more informa
tion on the subject of costs, at least splitting them up into steelwork
foundations, and decking.

He would like to see included an elevation of a part of the approach
spans; the type of girder was quite exceptional—12 feet deep and 200 fee
span—but no drawing was shown. He could not find out the spacing
the stiffeners or the spacing of the cross-girders. Was the concrete dec
rigidly attached to the stringers? It was in fact almost impossible t
place a concrete deck on stringers so that it did not make rigid contact, and
realizing that, in some bridges with which he had dealt he had made
definite mechanical bond and had made calculations assuming that th
composite structure formed the deck of the bridge. Had that been don
in the present case?

He understood that one of the hangers in the navigation-spans ha
fractured from vibration after the span had been erected. That inciden
was of exceptional interest, and perhaps the Authors would give a complet
account of it. There were many bridges with members of a similar form
to the hanger which failed in the Storstrøm bridge. They might not be s

long, and the conditions in which they were used might differ, but it was rather startling to find that a member of that character could vibrate so as to cause it to fail in the structure. It was most fortunate that that failure occurred before the bridge was put into service.

A feature of the design to which he would call attention was the accommodation of the Storstrøm bridge, in comparison with the practice which would probably have been adopted in Great Britain. Only a single line of railway was provided over a 4-mile length, and he found it difficult to imagine any British railway company building a major bridge of great length for only one line; in the case of the Storstrøm bridge, however, one line served the purpose. Again, the roadway had only two traffic-lanes for a fairly heavy traffic; the Paper stated that during the first year the traffic over the bridge was 1,000 vehicles per day. Those facts were of the utmost importance in connexion with the construction of the bridge, for had the bridge been planned to provide superior accommodation it would not have been built at all, as the money would not have been available. He rather felt that bridge-projects in Great Britain were for accommodation which was more extensive than it need be, and that the cost was correspondingly increased, so that bridges which might be built were not built. There was another advantage in building initially at the minimum of cost. The capital saved would accumulate and suffice to build another bridge when traffic had increased. The second bridge might well occupy a position somewhat different from that of the first, so avoiding a heavy concentration of traffic in one place, or possibly providing a service for a need which had developed since the building of the first bridge.

Another feature of the bridge was the size of the navigation-spans. Those spans were only 335 feet and 447 feet, but they were evidently sufficient, for it was stated in the Paper that 15,000 ships used the channel annually, and he believed it was a fact that none had come to grief by striking the piers of the bridge. Those openings, however, were very different from the kind of opening which was expected on an estuary in Great Britain by the average British navigator. To judge by experience in Parliamentary proceedings in connexion with bridges, the mere idea of a bridge across an estuary seemed to upset navigators completely. They wanted a bridge which crossed the river in one span, or at any rate had spans far larger than had sufficed in the case of the Storstrøm bridge. He hoped that the fact that the navigation-spans provided in the Storstrøm bridge had proved sufficient might lead to a different attitude towards new bridges in Great Britain, for the length of the span might make a great difference to the cost of the bridge.

Judging by what seemed to be the accepted practice in Great Britain, the most striking fact about the whole structure was that it was a bridge at all. In Great Britain, although no money could be found for bridges, tunnels continued to be built regardless of expense. The site of the

Storstrøm bridge was ideal for a tunnel, and it was pleasing to find that spite of that the Danish Government had decided to build a bridge. tunnel would have cost at least twice as much as the bridge, and man times as much per annum for maintenance, and he felt that the accepte practice in Great Britain required some explanation.

Mr. Conrad Gribble remarked that no bridge had been built in Grea Britain in any way comparable to the Storstrøm bridge since the preser Tay bridge was constructed, and it was interesting to consider how enginee ing knowledge had progressed in the last 50 years between the design of th Tay bridge and the time when the Storstrøm bridge was designed. The was some general similarity between the two structures.

The first point of difference was the use of high-tensile steel in th Storstrøm bridge. The limit of nearly 15 tons per square inch that ha been adopted for certain combinations of stress in the girders was somewha higher than was usual in Great Britain, and showed the confidence whic the designers had in the material. Mr. Gribble did not see any very clea reason given for the three types of steel used, namely, the "Chromador high-tensile steel, a superior form of mild steel (actually a mangane steel), and an ordinary steel. He assumed that since it was not alway possible for practical reasons to reduce the sections to those which coul theoretically be adopted for the various types of steel, the practice was t use the high-tensile steel when it could be stressed up to the maximu figure, then to use the manganese steel when the high-tensile steel coul not economically be used, and finally, for those sections which could no be stressed in any case to more than a nominal figure, the mild steel wa used. He would welcome a little more information on that matter, a it was rather unusual to employ those three types of steel in one structur He assumed that no difficulty had been experienced in the design on accou of the greatly increased deflexion obtained when using a stress of nearl 15 tons per square inch on the steelwork.

Another modern method of construction, evolved in comparativel recent years, was the successful use of under-water concreting. Th Authors referred to the mix of 1 : 2½ : 2½ as being one which did not give particularly strong concrete, and some explanation of the reasons for usin that mix would be useful. In similar work with which he had been con cerned, less sand had been used than that, but very satisfactory results with a very strong concrete, had been obtained.

There was one small allusion to vibrated concrete in the Paper. Tha again was a system of construction which was unknown when the Ta bridge was built. The Authors did not seem to have used it to any grea extent except for the footpath-slabs, which were presumably pre-cast there would be no particular difficulty in using vibrated concrete for suc slabs, but where the concrete was cast in situ it would be very difficult t vibrate it.

With regard to the design of the structure, presumably there were certain disadvantages, as well as advantages, in the use of continuous spans. He imagined that the articulated joints added very considerably to the cost of the work where continuous spans were adopted; whether or not that was proportionately a serious matter would no doubt depend upon the length of the spans that were used. Nothing was said in the Paper regarding the economics of that form of construction compared with simple spans of 60 metres, and it would be of interest to know whether the Authors had any information on the subject. With regard to the plate-girders themselves, he did not quite understand the statement that it was desirable to use angles as large as 12 inches by 12 inches; he thought that all the rivets necessary could have been arranged in 8-inch by 8-inch angles.

He was glad to see that the girders had been sand-blasted before painting, because he had always been of the opinion that that would be the solution of a great number of maintenance-problems with regard to painting. The cost of sand-blasting a large bridge had generally been considered to be beyond anything which could be justified, and it was therefore interesting to see that the estimated cost was only about 3*s*. a ton, which was not at all an unreasonable figure. It would be of interest to have details of the plant used and to know how successful the method had been.

Mr. J. Guthrie Brown referred to the bascule-span of the Masnedsund bridge, mentioned on p. 404. The pivoted or trunnion bascule type of bridge, despite the example of the Tower bridge, had not often been adopted in Great Britain, most opening bridges having been of the rolling-lift bascule, or Scherzer, type. The rolling-lift bridge, especially in the case of twin leaves, had the great advantage of giving sufficient clearance for navigation by partial opening much more rapidly than a swing-span, where full opening was necessary. A single-leaf trunnion bascule had of necessity to be almost fully opened for the passage of vessels. Had the Scherzer-type bascule been considered for the opening with which they had to deal? If so, what special reason had led to the adoption of the trunnion type? The width of the pier shown in Figs. 2, Plate 1, was about 40 feet, which would have been suitable for the rolling-lift type, although the foundation loads for a Scherzer-type bridge were generally much more severe.

The use of an alternating-current motor for the operation of the bascule, with a direct-current motor of similar size as a standby, was most unusual. Generally speaking, direct current had some advantages for the control and operation of moving bridges, and it would be of interest to know the reason for the adoption of alternating-current as the power-supply. For a very large Scherzer bridge which was at present being designed by his firm, the operating power would be supplied entirely by two diesel-engines

generating direct current, the alternating-current Grid-supply bei relegated to the duties of lighting and heating only. The provision of independent power-supply for a moving bridge, dealing with a large amou of river-traffic which could not be delayed, had much to be said in favour for both economic and strategic reasons.

He was interested to learn that the whole bridge had been coated wi aluminium paint. That paint had been used extensively in the Unit States and in Canada for many years, but he was under the impressi that the first time that it had been tried in Great Britain for a large brid was in 1936, when the Kincardine-on-Forth bridge was so painted. Te over 2 years had shown the advantage of that type of paint for the atm spheric conditions which had to be encountered at Kincardine, and t interval of 3 years since it was applied on the steelwork had confirmed th Had tests been carried out on the Storstrøm site before the decision adopt aluminium was arrived at? What were the undercoats? For t Kincardine bridge the four coats were red-lead paint, graphite pair graphite-aluminium, and aluminium; the first three coats were appli with a brush and the final coat by a spray—the opposite of what was do in the case of the Storstrøm bridge. The reason for the final spray-coat Kincardine was to enable a uniform colour to be obtained, as it was foun that aluminium applied by brush to the large areas of the plate girders w very streaky, largely due to the materials tending to segregate unless th were constantly stirred. Spraying had the advantage of ensuring ver intimate mixture. Perhaps the Authors would give the reasons for the method of applying the paints.

Mr. Ernest Bateson observed that one point which should be not in connexion with the approach-spans was that, with the adopted type span, the longitudinal forces due to tractive effort or braking effect two spans had to be carried by one pier, whereas with separate simp spans it was possible alternately to fix and free the bearings on opposi ends of the girders, and in consequence each pier only carried the long tudinal force on one span. In regard to maintenance, he would sugge that, even if the area to be protected in the case of plate-girders were twi that of lattice-girders, the maintenance of the plate-girders would st be a very much simpler proposition. It was a very simple matter to prote large areas; in fact, if the edges were looked after the flat surfaces too care of themselves. Lattice-girder spans, on the other hand, had man edges and only small flat surfaces, whilst batten-plates and lacing ba provided innumerable corners and crevices which could not easily b protected.

The navigation-spans were of rather an unusual type of tied arch, th stiffening girder being extremely heavy and taking the major portion the vertical loading in bending, whereas, generally speaking, the ties we relatively flexible, and the compression was taken in the arch itsel

Possibly the use of the very deep stiffening girder was dictated by the method of erection, as the girders had to span half the total width of the opening as simply-supported spans. Very light upper lateral bracing was used, and was almost entirely concentrated at the upper flange level of the arch-rib. Presumably, to have used lateral-bracing members of more nearly the depth of the rib would have involved more material by necessitating the use of laced members, and thereby complicating maintenance; it seemed to him, however, that the contribution of stress in the arch-rib was rather concentrated in one corner of the rib section, and if the shallow bracing were considered unavoidable, he would rather have seen it situated nearer the centre of gravity of the rib itself. The forces from the lateral system were assumed to be transmitted through the portal-bracing, which was placed on the end suspender, and transmitted through the floor-system to the support. The adopted arrangement set up a statically-indeterminate condition. Forces usually took the nearest path to the point of anchorage, and he would suggest that a considerable amount of force was bound to be transmitted down through the arch-ribs, which were considerably stiffer than the portal-bracing. Had that point been investigated?

He liked the use of the twin roller bearings, as compared with the English practice of using a large number of small rollers, which had to be accurately machined, ground, and polished to very fine limits, to ensure an even distribution of the load, and which, when they were brought into use, were left exposed to the elements. It seemed to him that if that type of bearing were adopted it should be properly protected against corrosion and the intrusion of dirt, because it was physically impossible to clean it in service. In the case of the Storstrøm bridge the use of the twin rollers very largely eliminated most of those difficulties.

It was very difficult in a work of the kind in question to know to what extent the designer had a free hand. The designer had very often to work to a specification with which he was not in complete agreement, and was often pilloried for doing so. So far as the provision for painting was concerned, he thought that that was probably put forward by the constructors of the bridge, and they were to be congratulated on making such adequate provision for painting and maintenance, although he had to admit that he was rather astonished at the figure of 630 tons of steelwork for gantries and runways, even though the material was very well distributed throughout the whole structure. If full use were made of the facilities provided it was, no doubt, a case of money well spent.

What were the first three coats of paint? He noticed that they were sprayed, and that suggested to his mind that they were not of the heavy-bodied type of priming paint, such as red lead, which was customary in Great Britain. The fourth coat was applied with a brush, that, as Mr. Guthrie Brown had remarked, being the reverse of what was usually done.

Mr. Bateson's experience had been that aluminium paint applied over a previous coat of aluminium tended, owing to the vehicle used, to pull up the undercoat; he believed that that was the reason why spraying was adopted for the aluminium finishing coat on the Kincardine bridge, as, the third coat being aluminium, it was found impossible to produce a satis-factory finish by applying the fourth coat with a brush. Incidentally, when several coats of aluminium were applied, it was very difficult to dis-tinguish between the coat already on and the coat which was being applied, and in the case of the Kincardine bridge a colour-distinction had been used. Had anything of that kind been done in the case of the Storstrøm bridge?

With regard to the extra cost of sand-blasting, he regarded the figure quoted of 3*s*. a ton as rather astonishing. Assuming an average thickness of metal of slightly over 1 inch, that represented about 80 square feet per ton, because both sides of the metal had to be treated, and the cost worked out at something like ½*d*. per square foot. He would like very much to know exactly what the cost covered, because, obviously, with sand-blasting a considerable amount of expenditure was incurred in providing the necessary power for operating the blast. He was surprised to note that the sand-blasting was found ineffective for removing mill-scale. He had seen demonstrations of sand-blasting, and no difficulty had been found at all in those cases in removing the mill-scale, whilst small unsuspected particles of metal which had been rolled into the surface had also been removed, leaving slight depressions.

Mr. F. M. Fuller said that the lengths of the anchor-arms and sus-pended spans in the cantilever approach did not appear to agree with the generally-accepted economic limits. It might be that having selected the plate-girder design, the minimum sections, such as the 12-inch by 12-inch angles, necessitated an alteration in the usual economic limits, as he thought that it would be agreed that with a lattice-design a far greater variation in girder-section could be obtained for given conditions.

The roller- and knuckle-details were of interest to English designers. He had not seen a bridge before with spans which were supported at both ends by roller bearings and in which the wind-bracing was relied on for fixity. In view of the fact that the rollers for the suspended spans were 3 feet 2 inches in depth, and for the 400-foot span somewhat less, he assumed that the Authors' principles were the shorter the span the bigger the roller, but perhaps they could explain that point. He would also like to refer to Mr. Ernest Bateson's statement that it was not normal English practice to design bridges with twin rollers. The next bridge which would be opened over the Thames, under the direction of Mr. Peirson Frank, M. Inst. C.E., would embody that detail, possibly not for the first time.

He wished also to refer to the working stresses given in the Appendix to the Paper. The stresses for high-tensile steel seemed to agree with

the accepted values for British practice, with the exception of the shear in girder-webs; 0·8 times the tensile stress gave a permissible stress of 10 tons per square inch. Did the Authors have the courage to work to that figure? If not, perhaps they would give the maximum stress to which they actually worked.

Mr. W. T. Everall observed that the Paper was of particular value as it dealt with many features of the design for a large bridge built of high-tensile steel. In the Appendix a specification was given for the steels used, as well as many of the rules and regulations on which the design was based. It would be of interest to know whether the code of practice for the use of high-tensile steel in bridges was adopted by the Danish State Railways before the project was started. He mentioned that because at the present time a sub-committee of the British Standards Institution was drafting a specification for the use of high-tensile steel for bridges, and there might be relevant information in the Danish specification suitable for adoption in the British. For instance, the Danish specification provided for batten-columns as well as for lattice-columns. The existing British specification for bridges did not refer to the use of battens. It seemed a subject worthy of attention, especially as the use of welding and riveting in composite structures was rapidly developing. A clause defining the adoption of batten-columns would be a valuable addition to the specification.

Reference was made on p. 409 to high-tensile steel rivets and to the satisfactory results obtained by their use in fabrication, there being no departure from normal shop processes. He was interested to know of that, because some difficulty had been experienced in the past in obtaining sound rivets for long grips when using that steel. He had had an opportunity, when visiting San Francisco in 1935, of inspecting the work on the Oakland Bay and Golden Gate bridges, then under construction. He had noticed that some of the rivets of the bridge had grips of from 10 to 12 inches. An engineer of the Oakland Bay bridge staff told him that a special technique had been developed for closing the rivets, and that at one time there had been some difficulty in developing a suitable high-tensile steel rivet. Unfortunately, he was not able to see the method of closing the long rivets demonstrated, and he would be glad if the Authors would give the maximum grip used with the 1-inch rivet and also the chemical properties of the high-tensile rivet steel, which were not specified in the Appendix.

With regard to the design of the steelwork, there was one matter which he would like to discuss and to which Mr. Bateson had already referred. It was in connexion with the top lateral bracing system of the navigation-spans. On p. 403 the statement was made that "The arch-ribs are connected by a system of double diagonal lateral bracing without cross members, . . ." From the details shown in Fig. 8, Plate 1, it would appear that the members consisted of rolled steel joists 10 or 12 inches in depth. It seemed to him that owing to their length they had very little stiffness

value. In view of the loading conditions, he suggested that it would ha[ve] been better to have made the bracing members the full depth of the boo[m]

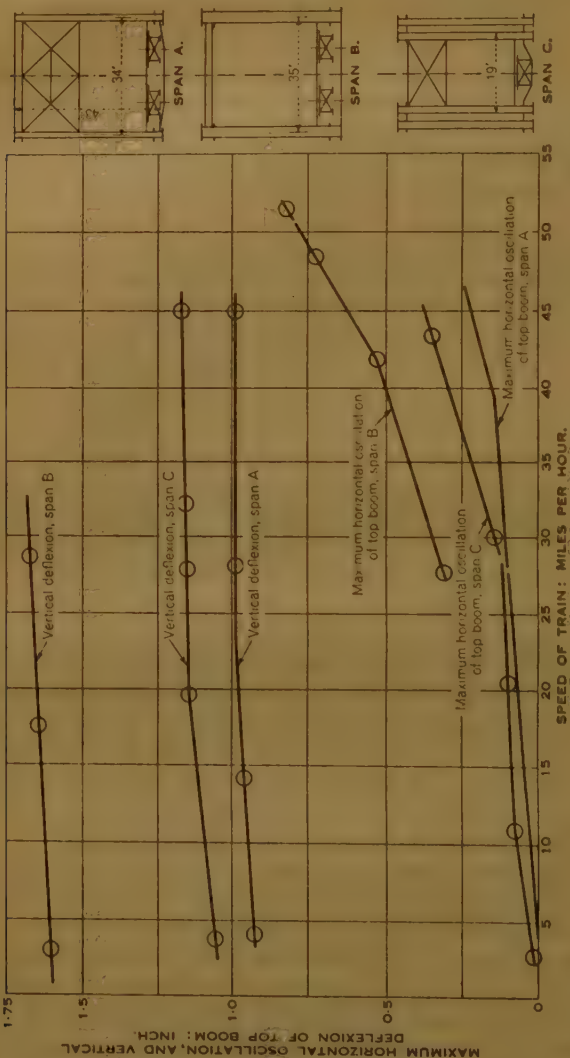


Fig 34.

SPAN A: 250-foot double-track through span, with robust top laterals, cross frames, and end portals.
SPAN B: 300-foot double-track through span, without laterals and end portals (having only cross frames).
SPAN C: 360-foot single-track through span, with top laterals, cross frames, and end portals.

and of considerable rigidity. The slenderness-ratio of the main arch-rib made the provision of adequate bracing necessary.

In considering the loading of the railway track, which was on one sid[e] of the deck, the assumption made was equivalent to a live load of abou[t]

800 tons on the 450-foot span, unequally distributed between the two arch-ribs. The dynamic effect of that train might produce considerable horizontal oscillation of the top boom if the latter were not adequately braced.

Although his experience did not include bridges of that type, he had partly designed and erected, and tested, several large railway bridges containing spans of up to 360 feet. It might be of interest to show the results of observations, taken on three large bridges with different systems of top bracing, in connexion with measurements of vertical deflexions and horizontal oscillations of the top boom. It would be seen from *Fig. 34* that, whilst the vertical deflexions showed little change as the speed increased in all the three types, there was a rapid increase of the horizontal oscillations of the top boom in span B as the speed increased. The bridge consisted of spans of 300 feet, and had only cross frames attached to the stiff vertical posts forming the top bracing system, reliance for the bracing being placed on heavy cross frames attached between the stiff vertical struts and the railway floor-system framed between the bottom booms. Type A, which might be considered a normal well-braced bridge, was provided with a robust system of top laterals, cross-frames and underportals, and showed comparatively little change in the development of horizontal oscillations as the speed increased. He ventured to suggest that those results indicated the importance of adequate top bracing in large-span bridges, chiefly on account of maintenance-conditions, because he considered that a slender form of bracing would induce loose rivets in time, and would probably cause the members to go out of action and require renewal. Similar information to that shown in *Fig. 34* was usually called for by the Senior Government Inspector of Railways in India for all large bridges before the spans were opened for use by the public. It would be of interest to know whether or not deflexion- and oscillation-observations, under reasonable capacity loads and speeds, had been recorded on the Storstrøm bridge.

Mr. Cyril Parry observed that the question of type of, and material to be used for, forms, received very careful consideration, and owing to the considerable number of uses, coupled with the exposed position, influenced the decision to use practically all-steel forms. At the outset there was a tendency to consider a system of construction of the piers in 0·8-metre lifts by using two bands of shutters, but an alternative scheme for the concreting of the piers with a single band of shutters in 1·6-metre lifts was approved by the Contractors, Messrs. Christiani & Nielsen, when they were satisfied that a good job could be made of, and with, the forms. The number of forms supplied was sufficient to enable work to proceed simultaneously on two normal piers and one of the large piers for the main openings. The material for those large piers was to be convertible to permit it to be used on normal piers afterwards.

The method of restraining the bursting pressure of the concrete on the lower edge of the forms was by means of tapered screw anchors [1] (*Fig. 2*, p. 415). Those anchors eliminated the necessity for either tying through the concrete, or using embedded nuts and plates. Tests were carried out on site to ascertain the safety of the device under service-conditions.

The forms were made from $\frac{3}{16}$-inch plate suitably stiffened with angles and braced with horizontal and vertical channel walings. Special precautions were taken to obtain a clean line on the horizontal construction joints. The external sides and end forms were each mechanically handled in one piece from the extended corners of the well-forms. The end forms were made wide enough to deal with maximum-width piers, but the side forms were exact length. That avoided alteration to the end forms for dealing with the taper on the piers. The internal taper faces of the wells were shuttered with steel-framed timber forms. An external working stage was incorporated in the forms, thus dispensing with any form of scaffolding.

The forms were made in England from British steel, and he understood that they behaved well and produced satisfactory results.

The Authors, in reply, observed that they would not like the impression to be gained that the Storstrøm bridge was an expensive bridge; as a matter of fact, they thought that it was a very cheap bridge. They had certain figures showing the relative cost of other bridges recently built which they had obtained from figures published from time to time; it had to be borne in mind that the figures were only generally indicative, and that the differences in time, venue, topography, loading and type between one bridge and another made strict comparison impossible. The bridges below mentioned were of the multi-span type, none of the spans being extremely large. The cheapest bridge that they had been able to find was the Kincardine-on-Forth bridge, a low-level road bridge with an approximate length of 2,400 feet, there being twenty-seven spans, the average width being 89 feet, and the price £2·25 per square foot of finished bridge-surface. Then there was the Burrard bridge at Vancouver, a high-level road bridge with a length of 2,800 feet, with thirty-one spans averaging 90 feet, price £2·70 per square foot. Next came a bridge with rather bigger spans, the Khedive Ismail bridge at Cairo, a low-level road bridge with a length 1,254 feet, consisting of eight spans averaging about 160 feet, at a price of £3·60 per square foot. The Storstrøm high-level road and rail bridge had a length of 10,535 feet in fifty spans, the average length being 211 feet, and the price was £2·75 per square foot, which was very nearly the lowest price. In view of the fact that the spans were considerably bigger than in the other cases which they had mentioned, they thought that that was a very low price for a bridge of that character. The Little Belt bridge in Denmark, a

[1] British patent No. 410855.

high-level road and rail bridge, was 3,866 feet long in thirteen spans, the average length being 298 feet and the price £4·65 per square foot. The Lake Champlain bridge in the United States of America was a high-level road bridge 2,187 feet long, in fourteen spans averaging 156 feet, the price £2·92 per square foot. The Lower Zambezi bridge was a low-level rail bridge 8,662 feet long; it consisted of thirty-three main spans, averaging 262 feet, the price being £5·88 per square foot, whilst the seven secondary spans, totalling 1,155 feet and averaging 165 feet, cost £5·61 per square foot.

Mr. Gribble and Mr. Bateson referred to sand-blasting. It had to be admitted that the extra price for sand-blasting the Storstrøm bridge over and above the price originally offered for scraping it was extremely low, and they believed that that was due to the fact that the Danish sub-contractor who was given the work had had previous experience on the Little Belt bridge, and there were reasons, perhaps not altogether connected with the cost of the sub-contractor, which affected the price, so that the figure given might not be quite a true criterion.

Mr. Gribble referred to the mix of the under-water concrete. The natures of the coarse and fine aggregates which were used were such as to form a mixture in which there was not, in fact, a very large excess of sand; there was at least 10 per cent. in excess of what was required to fill the voids, but the excess of sand was not so great as the proportions mentioned might appear to indicate. The concrete was examined after it had been deposited and was found to make an extremely homogeneous substance under water, and there was very little laitance. Vibrated concrete was not used very much on the bridge, one reason being that in Denmark the custom—and everything was ruled to a large extent by custom—was to mix the concrete very wet. That was probably not at all a good custom, and the use of very wet concrete, with a slump exceeding 4 inches, made the use of vibrators not only unnecessary but positively detrimental.

It should be made clear that the engineers of the Danish State Railways, and in particular Dr. Anker Engelund, were responsible for laying down the type of structure to be used, and in some cases for the details, so that a number of the questions which had been raised were not questions to which the Authors could give an exact reply.

In reply to Mr. Freeman's query regarding pier-settlements, so far as the Authors were aware, there had been no appreciable settlements of the bridge-piers since the completion of the work, but in the event of settlements occurring provision had been made in the design for the girders to be jacked up and packings inserted.

With regard to the relative economy in first cost of plate- and lattice-girder designs, the figures for a series of 70-metre simple lattice-girder spans had been quoted for comparison with the plate-girder design because in that case a complete design of lattice-girders in 70-metre spans was

available. The figures demonstrated that even when a 70-metre lattic girder span was compared with the 60-metre (average span) plate-gird as built the advantage rested with the former. Mr. Freeman was, of cours right in supposing that a 60-metre lattice-girder span would probab show a greater economy still in favour of the lattice design. A sing average price had been quoted for the whole of the structural steelwo For the approaches the quantities per foot of bridge were :—

Main girders	1·20 ton
Lateral bracing and portals	0·10 ,,
Cross girders	0·21 ,,
Stringers	0·19 ,,
Mix steelwork	0·11 ,,

The total cost of the Storstrøm bridge proper, excluding the Masnedsu bridge and all approach works, amounted to £99 per foot.

The average costs per foot of the plate-girder approach-spans of th main bridge, based upon the schedule rates, were :—

	£
Structural steelwork	48·3
Steel castings	2·6
Reinforced-concrete decking	5·2
Road and footway surfacing	0·9
Concrete in piers and foundations	11·3
Granite in piers	1·8
Steel piling to piers	10·6
Excavation	1·6
Unallocated expenses	10·8

It had to be borne in mind that certain materials (for example, cemen and rail track) and works were undertaken direct by the State Railway and were not included in those rates.

With regard to the failure of some of the navigation-span hange which had taken place during construction, the facts were :—the hange as originally designed were all of I-section, consisting of a transverse we plate 20 inches by ½ inch and four angles 4 inches by 4 inches by ½ inc During the construction of the bridge and before the laying of the concre deck had commenced the outstanding legs of all four angles of one of th longest hangers of the railway girder of the centre navigation-span we found to be fractured near the edge of the top gusset. The web-plate an the legs of the angles connected to it were not affected. An investigatio indicated a close agreement between the natural period of vibration the longest hangers at that stage and that of the complete girder. Co siderable vibration of the hangers under wind had been observed. Short afterwards, following a severe gale, similar fractures were found in tw more hangers of one of the side navigation-spans, and in that case also th natural periods of the hangers and of the whole span were found to agr closely. The lengths of the longest hangers of the centre and side navig

tion spans were 67 feet and 47·5 feet respectively. Investigation failed to reveal any trace of weakness in the material of the hangers, and the failures were ascribed to fatigue under prolonged vibration, set up mainly by wind. To overcome the effect of the flexibility of the hangers, all the longer members of the centre span were replaced by sections consisting of two battened channels spaced 20 inches apart with their flanges turned inwards. The longer hangers of the side navigation-spans were reinforced by the addition of a T-section on each side of the web with flanges outstanding, connected by cleats at intervals.

The working stress adopted for the "Chromador" steel for dead-load, live-load, and impact stresses was 12·7 tons per square inch, and the higher figure referred to by Mr. Gribble was only employed for combinations of those loads with stresses set up by wind, traction-forces, etc. Actually the first combination of stresses was the governing one. The specified figure for permissible shear stress of 80 per cent. of the value allowed in tension was the usual one in Continental specifications, and was the same as that allowed by the German State Railways and in the French Standard Specifications. In reply to a point raised by Mr. Fuller, the maximum shear stress in the webs of the approach-girders was actually 5½ tons per square inch. The question of the permissible shear stress was too closely bound up with that of stiffening to be considered independently, and in that case the critical strength of the web against buckling under both shear and bending was examined, and the spacing of both vertical and horizontal stiffeners proportioned accordingly. The specification adopted by the Danish State Railways for that work was modelled on that used for the Little Belt bridge, which was also constructed in high-tensile steel. The specification was similar in several respects to that of the German State Railways. High-tensile steel had been used very extensively for both rail and highway bridgework on the Continent for the last 10 years.

The reason for the adoption of three grades of steel was that the specified yield-point of 18·4 tons per square inch could not be obtained with ordinary mild steel to the British Standard Specification, and a special material had, therefore, to be provided. Ordinary mild steel was used only for lightly stressed parts. The cost of the high-manganese steel adopted was less than the "Chromador" steel, and it was found economical and convenient to use it for many parts which, if made from "Chromador" steel, would have proved unduly light.

The economics of the cantilever form of construction adopted had been referred to, and, whilst no figures were available for a direct comparison, it was considered that the economy in total cost in the present instance was probably not less than 3 or 4 per cent. The joints at the ends of the suspended spans did not present very great difficulties, and they were in any case cheaper than the extra end bearings required for simple spans. A considerable economy with the cantilever type of construction could be

effected in the width of the pier-shafts when only a single bearing instead of two had to be accommodated on each pier. No difficulty was experienced in providing for the expansion-movement at the piers with the single rocker bearings adopted, or for dealing with the longitudinal forces at the foundations of the piers carrying the fixed bearings.

The lengths of the suspended spans in relation to the cantilever-arms were arrived at with a view to the moments at the centre of the suspended spans being as nearly as possible equal to those at the supports. In that connexion, it had to be remembered that the girders were of parallel construction with web-plates of uniform thickness. The economic ratio of length of suspended span to cantilever opening was approximately 68 per cent., whereas a proportion of 71 per cent. was adopted. The lengths of the anchor-spans were chosen to give moments at the centres of the girders not greatly in excess of those at the supports.

The method of supporting the suspended spans certainly differed from normal practice, but it was clearly desirable, if undue movement at deck level due to the deflexion of the span were to be avoided, that the point of articulation should be as high as possible. That had been achieved by fixing one end of the span in the plane of the top laterals and allowing for movement by means of rocker bearings at all points of vertical support. The proportions of the rockers were arrived at from a consideration of the depth required for the cantilever projections at the ends of the suspended span and cantilever-arm, and the width of bearing available.

Whilst agreeing with Mr. Guthrie Brown that the fixed trunnion type of bascule might have been prejudiced in Great Britain by the appearance of the Tower bridge, it was nevertheless a fact that its selection for use in the Masnedsund bridge by the Danish State Railways was largely dictated by a feeling for the appearance of the finished bridge. Actually the earlier designs had been for a Strauss type of bascule with overhead counterweight, which was a very ugly type and had been discarded. The rolling-lift type had also been considered, but had been abandoned, partly because the foundation loads in that type of bridge were more severe than for the fixed trunnion bascule, but mainly because its appearance was difficult to harmonize with any other type of construction. The unusual arrangements for the power-supply in the present case were dictated by local conditions which would not normally apply.

The State Railways were also probably guided by æsthetic rather than by strictly economic considerations in their choice of the type of navigation-spans. The actual weight of steel in the arches as constructed was approximately 13 per cent. greater than would have been required for a normal type of through lattice-girder of cantilever construction upon which the Contractors' tender was based. The actual weight of steel in the three spans as built was 3,460 tons, as compared with an estimated weight of 3,060 tons for the lattice-girders.

Attention had been drawn by Mr. Bateson to the type and position of the lateral bracing of the arch-ribs. It had to be borne in mind that the area exposed to wind by the arch-ribs was extremely small, and that in consequence the wind loads in the diagonals were very light indeed, the most important function of the laterals being the effective bracing of the arch-ribs as compression-members. The arch-rib sections had a solid top flange-plate and were connected on the underside only by lacing, so that a very much more rigid attachment was provided in the plane of the top flange. Some difficulty always arose in the design of the portal-bracing at the ends of through spans due to the alternative paths available for the shear to reach the bearings. Actually the possibility of the shear being carried by the end sections of the arch-ribs in the present instance had been investigated, and provision had been made accordingly.

So far as the Authors were aware, no records of lateral oscillation of the arch-ribs were available, but the whole bridge appeared to be extremely rigid and free from vibration under traffic. The fact that the bridge carried a heavy concrete road and footpath in addition to the single railway track made the dead-load proportion considerably greater than would be the case with a normal double-track railway bridge. The fact that the arch-ribs were continuous from end to end of the bridge and not broken at the connexion of the end raker, as would be the case with the normal through truss, probably contributed to their freedom from horizontal vibration. Those are factors which mitigated the dangers to which Mr. Everall had drawn attention.

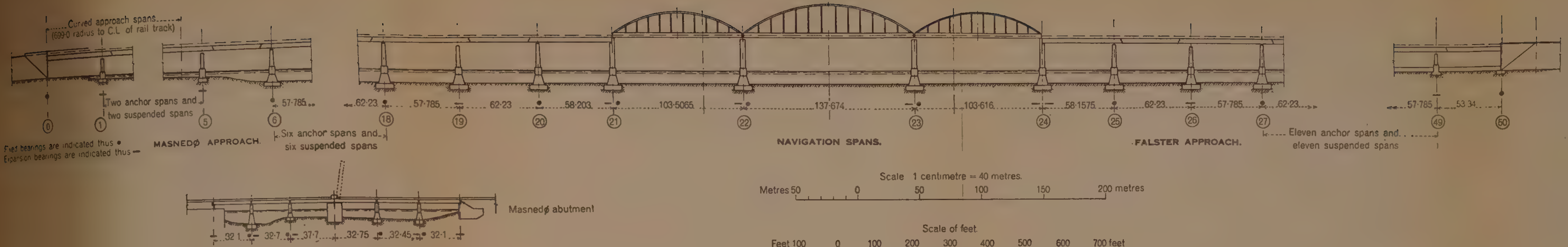
In connexion with the sand-blasting and the painting of the structure, commented upon by Mr. Guthrie Brown, Mr. Gribble, and Mr. Bateson, it should be explained that the Danish State Railways had specified for a 5-year guarantee period where the paintwork was concerned, and they were also very desirous that the sub-contract for the painting work should be sub-let to a Danish firm. The firm of S. Dyrup & Company, of Copenhagen, were at the time sand-blasting and painting the Little Belt bridge, and in view of the proficiency that they had gained it was thought advisable to entrust them with the work of sand-blasting and painting the Storstrøm Bridge also. They were invited to quote alternative prices for cleaning the steelwork by the usual scraping and wire-brushing process and for cleaning it by sand-blasting. In making up those alternative prices they no doubt took into consideration the repairs which they might have to do under the 5-year guarantee period, for which they in their turn had to assume responsibility. The smallness of the extra price which they actually quoted for doing the cleaning work by sand-blasting was probably due to the fact that they had more confidence in the durability of their paintwork if it were laid upon a clean sand-blasted surface than if it were laid upon a scraped and wire-brushed surface; possibly they might also have expected that the Danish State Railways would insist upon such a large

amount of scraping and wire-brushing being done in the first instanc as would in the long run be almost as expensive for them as treating with sandblast. The undercoat paint which was sprayed on was not a red-lea paint, the spraying of red lead being regarded as too dangerous to th health of the workmen.

*** The Correspondence on the foregoing Paper will be publishe in the Institution Journal for October 1939; the Authors, in their repl thereto, will deal further with certain points raised in the Discussion.— SEC. INST. C.E.

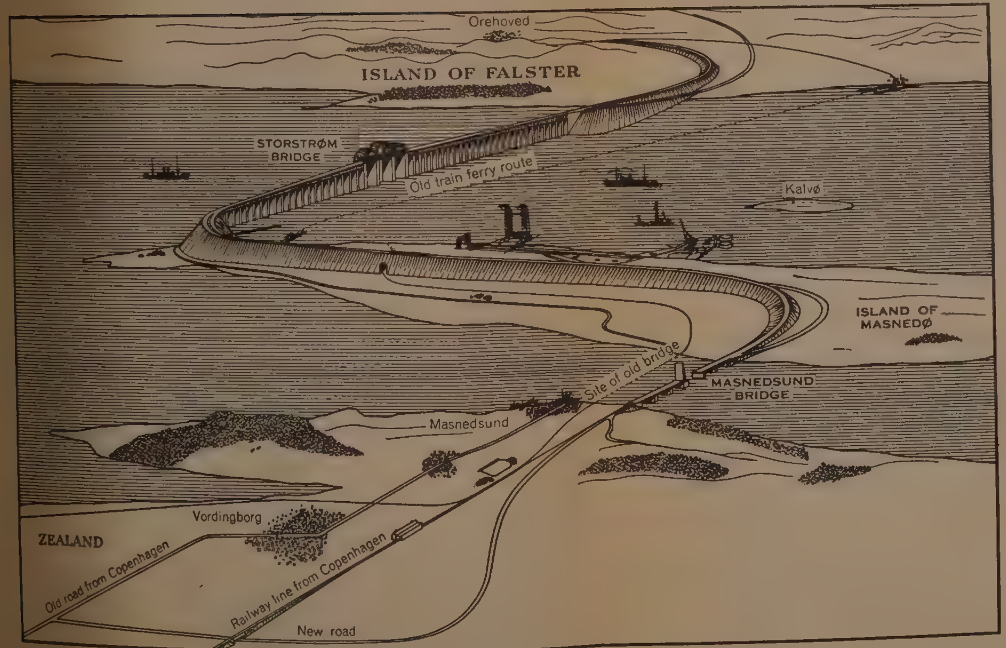
THE STORSTRØM BRIDGE, DE

FIGS: 2



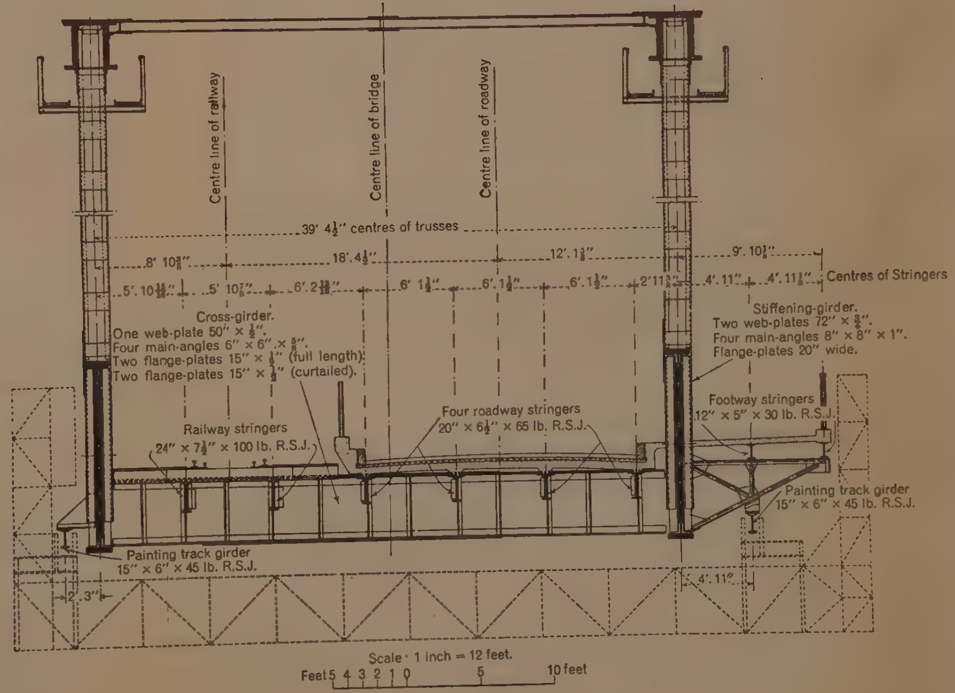
LAYOUT OF STORSTRØM AND MASNEDSUND BRIDGES.

FIG: 3.



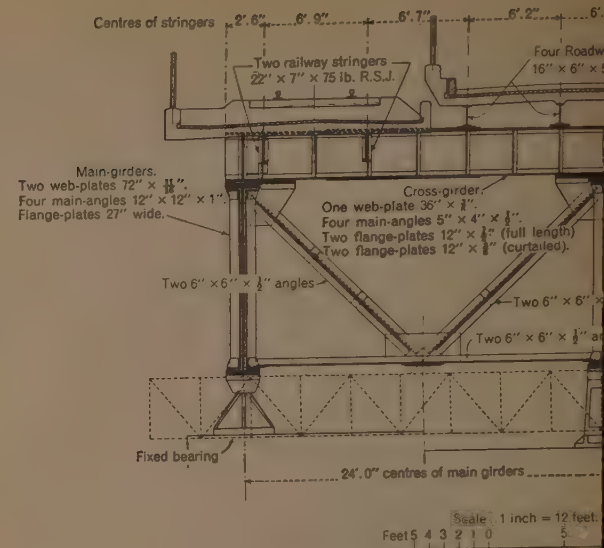
DIAGRAMMATIC PANORAMA OF THE STORSTRØM AND MASNEDSUND BRIDGES LOOKING SOUTH.

FIG: 8.



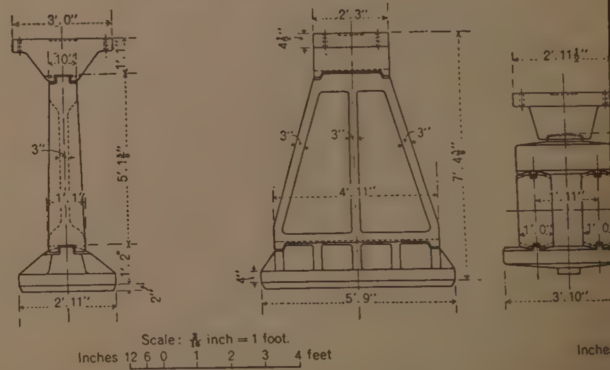
STORSTRØM BRIDGE: NAVIGATION-SPAN CROSS SECTION.

FIG: 9.



STORSTRØM BRIDGE: NAVIGATION-S

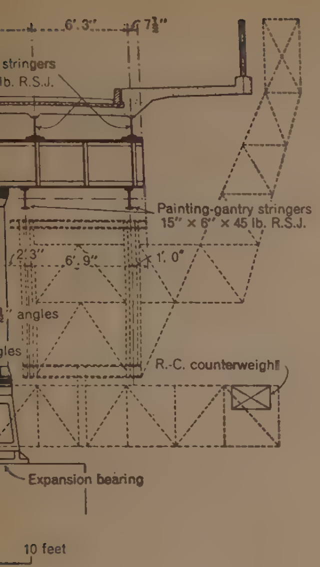
FIGS: 10.



APPROACH-SPAN EXPANSION-BEARING.

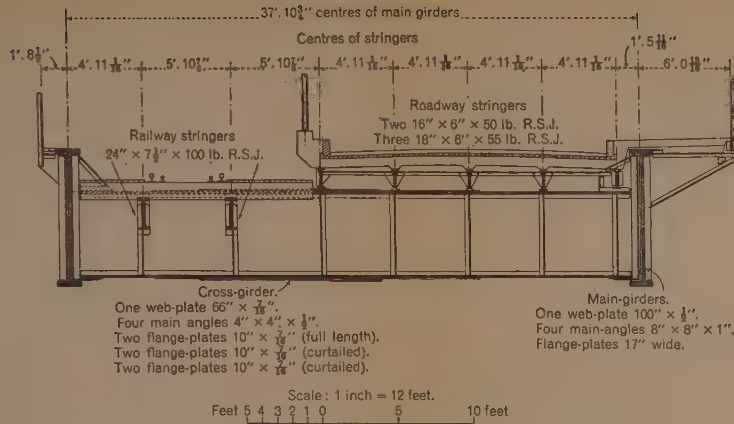
NAVIGATIO

William Clowes & Sons, Limited: London.



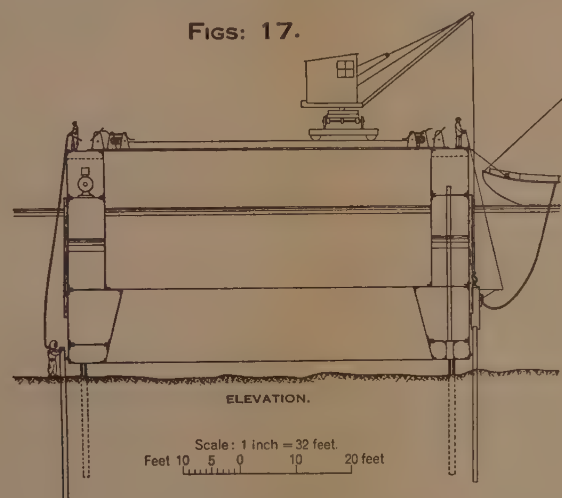
PAN CROSS SECTION.

FIG: 15.

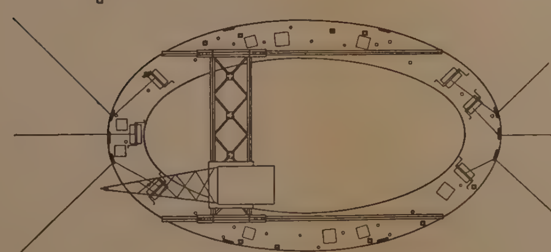


MASNEDSUND BRIDGE: FIXED SPAN CROSS SECTION.

FIGS: 17.



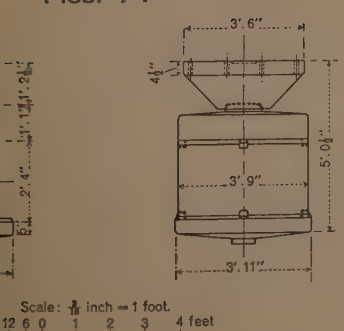
ELEVATION.



PLAN.

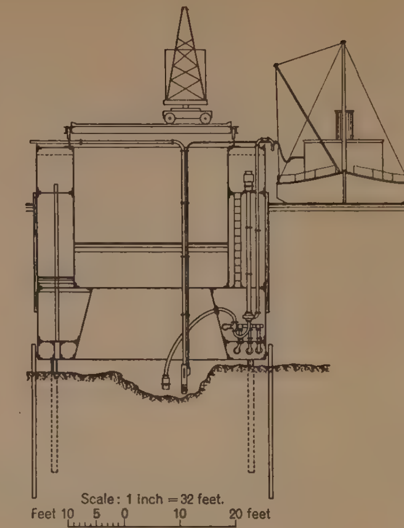
PIER-UNIT PLACING.

FIGS: 14.



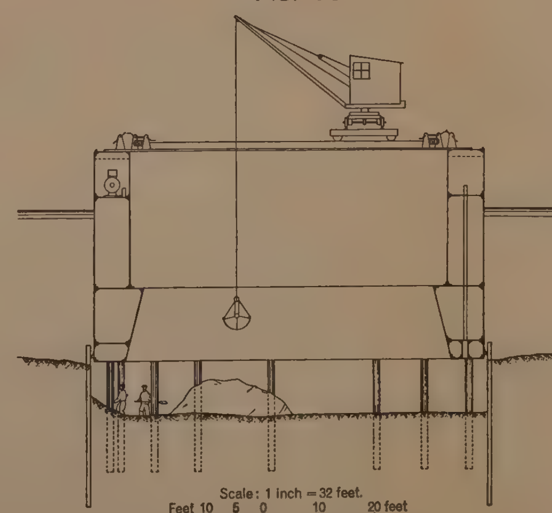
N-SPAN EXPANSION-BEARING.

FIG: 18.



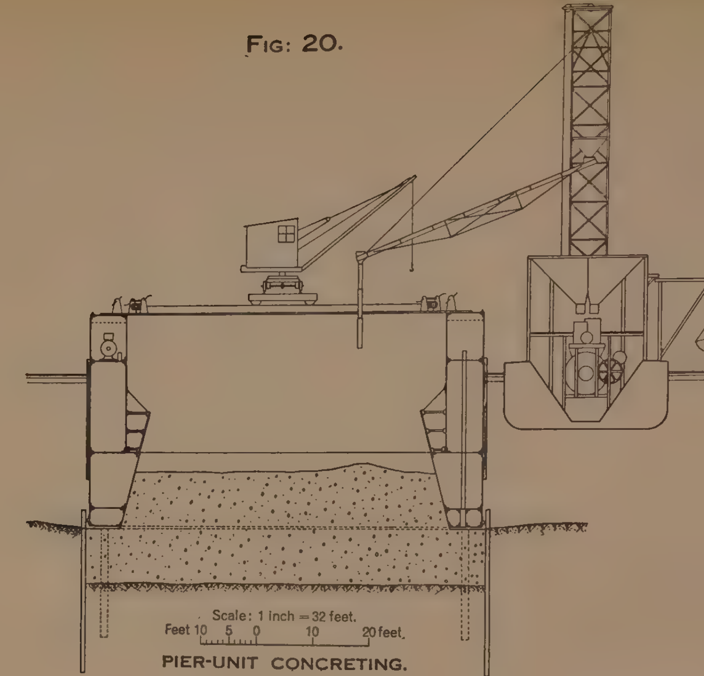
PIER-UNIT PUMPING.

FIG: 19.



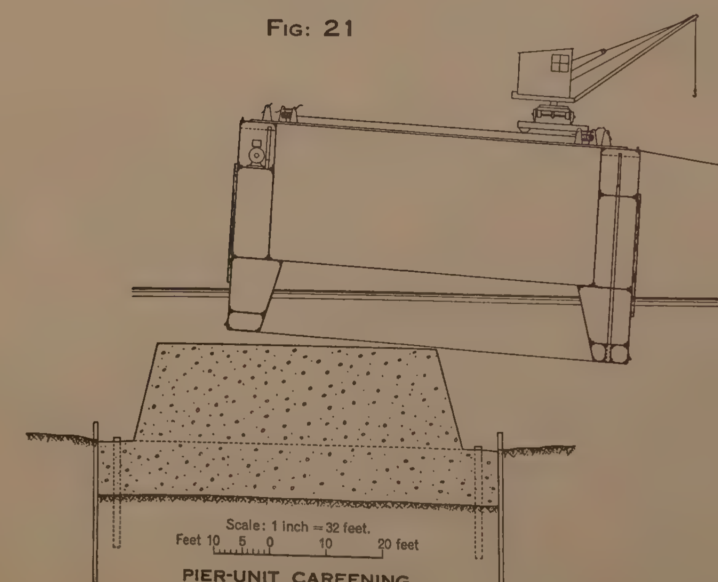
PIER-UNIT EXCAVATING.

FIG: 20.



PIER-UNIT CONCRETING.

FIG: 21



PIER-UNIT CAREENING.

G. A. MAUNSELL and J. F. PAIN.

ORDINARY MEETING.

7 March, 1939.

WILLIAM JAMES EAMES BINNIE, M.A., President, in the Chair.

The Scrutineers reported that the following had been elected as

Member.

Percy Stanley Robinson, M.C.

Associate Members.

William Thomas Anderson, M.C., B.E (*Sydney*).
William Frederick Armstrong, B.Sc. (*Cape Town*).
Frank Richardson Askin, B.E. (*New Zealand*).
John Frederick Bailey, Stud. Inst. C.E.
Eric Bayly.
Harold Beric Brown, B.Sc. (*Cape Town*).
William Marshall Chrystal.
Sidney Philip Cohen, B.Sc. (*Cape Town*).
John Michael Patrick Coogan, B.E. (*National*).
Robert Miller Mackay Cormack, B.Sc. (*Glas.*).
James Cross.
Neil Darroch, Stud. Inst. C.E.
Harry William Dupree, Stud. Inst. C.E.
Ian McKechnie Edwards, B.Sc. (*Glas.*).
Albert Edward Fallows, B.Sc. (*Bristol*).
Cyril Lloyd Ferreira, B.Sc. (*Cape Town*).
Hector William Forsyth, B.E. (*New Zealand*).
Wilfred Gilbert Freeman, B.Sc. (*Witwatersrand*).
Norman Charles Gallot.
Vernon Gordon Gilham, B.Eng. (*Liverpool*).
Colin Anson Gillett.
Fred Needham Green, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
William Green, B.Sc. (Eng.) (*Lond.*).
Frank Greenhalgh, B.Sc. (*Manchester*), Stud. Inst. C.E.
George William Haslam, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Robert Drake Hawkins, B.Sc. (*Bristol*).
Andrew Brodie Henderson, B.A. (*Cantab.*), Stud. Inst. C.E.
John Findlay Henderson, B.E. (*New Zealand*).
Samuel Hind Hodgson.
Arthur David Holland, B.Sc. (*Bristol*), Stud. Inst. C.E.
Lloyd Lindsay Hoskin, B.E. (*New Zealand*).
David Victor Isaacs, M.C.E. (*Melb.*).
Richard Henry Ardagh Johnson, B.A.I. (*Dubl.*).
George Officer Lawson, Stud. Inst. C.E.
William John Leith, B.Sc. (*Witwatersrand*).
John Dyson Marshall.
Francis Sumner Matthews, B.Sc. (*Bristol*), Stud. Inst. C.E.
James Vivian Metcalfe.
Solomon Simon Morris, B.Sc. (*Cape Town*), Stud. Inst. C.E.
Frederick Victor Murdock, B.A.I. (*Dublin*).
Neil William John Murray, B.Sc. (*Witwatersrand*).
Thomas Patrick Murray, M.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.

Eustace Misselbrook Oborne, B.Sc. (Eng.) (*Lond.*).
George Rawlings Pape, B.Eng. (*Liverpool*).
John Aitchison Patterson, B.Sc. (*Edin.*), Stud. Inst. C.E.
Laurence Pattinson, B.Sc. (*Leeds*).
John Howard Payne, Stud. Inst. C.E.
Cyril David Peat, B.E. (*New Zealand*).
James Harry Prior.
Edward George Russell, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Bernard Brooke Sampson, Stud. Inst. C.E.
Emil Archibald Shaw, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Matharu Ganga Singh.
Alleyne Arthur Smith, B.E. (*Ne Zealand*).
John Holmes Smith, B.E., B.Sc. (*Ne Zealand*).
Philip Roy Spencer, Stud. Inst. C.E.
Walter Harold Taylor, M.C.I. (*Melb.*).
Charles Joseph Tustin, Stud. Ins C.E.
Arthur Jephson Wallis, B.Sc. (Eng (*Lond.*), Stud. Inst. C.E.
Henry Robert Watts, B.E. (*Ne Zealand*).

The following Paper was submitted for discussion, and, on the motio of the President, the thanks of The Institution were accorded to th Author.

Paper No. 5189.

"Considerations on Flow in Large Pipes, Conduits, Tunnels, Bends, and Siphons." †

By James Williamson, M. Inst. C.E.

(*Abridged* *.)

TABLE OF CONTENTS.

Introduction.

The Institution of Civil Engineers has been privileged in recent years in having been presented with a number of Papers [1, 2, 3, 4] dealing with practical and experimental matters on the flow through siphons and on the very closely related subject of flow around bends, and a mass of data has been accumulated. The Author has on various occasions in the discussions raised points of hydraulic principle which it appeared necessary to take into account if accurate analysis of the complicated flow-conditions were to be made. A review of the data available in the light of hydraulic laws as he understands them leads the Author to certain conclusions

† Correspondence on this Paper can be accepted until the 15th July, 1939.—Sec. Inst. C.E.

* The MS. and illustrations may be seen in the Institution Library.—Sec. Inst. C.E.

[1] P. Davies, "The Maramsilli Reservoir Automatic Siphon Spillway." Minutes of Proceedings Inst. C.E., vol. 224 (1926–27, Part 2), p. 39.

[2] A. H. Gibson, T. H. Aspey, and F. Tattersall, "Experiments on Siphon Spillways." Minutes of Proceedings Inst. C.E., vol. 231 (1930–31, Part 1), p. 203.

[3] P. Davies, "The Laws of Siphon Flow." Minutes of Proceedings Inst. C.E., vol. 235 (1932–33, Part 1), p. 352.

[4] P. Davies and S. V. Puranik, "The Flow of Water through Rectangular Pipe Bends." Journal Inst. C.E., vol. 2 (1935–36), p. 83. (February 1936.)

regarding the flow in siphons which are at variance with many view hitherto expressed, but are consistent with the application of principles f flow in closed pipes or passages.

As flow in siphons is complicated by high velocity-head, by the existen of bends which introduce vortex-flow and increased kinetic energy, an by the occurrence of low-pressure conditions, particular attention is give to the related aspects of ordinary pipe-flow, that is, the selection of reliable formula for flow in large pipes, the investigation of the increase kinetic energy with increase of the relative roughness which occurs in pipe-flow, the increase of kinetic energy produced by vortex-condition the variation of pressure-head across a section having non-uniform flow the establishment of approximate rules for correlating losses in small bend to losses in large bends, and the determination of suitable roughnes coefficients applicable to a wide range of surface-conditions in metal an concrete pipes, masonry, brickwork and concrete surfaces, river- an canal-conditions, and tunnels of unlined, partially lined, and complete lined form.

The formula selected for flow in large pipes is found to lead with limits to the establishment of an approximate basis for correlatin coefficient of roughness with absolute roughness, and for determinin conditions of similarity.

The principles investigated are applied in a method devised for ca culating the flow in siphons, and examples of actual siphon designs a given.

Basic Formulas.

As the application of laws of flow in pipes is found to enter largely int the elements of the siphon-problem, it appears desirable to proceed with a approved formula of wide range as a basis.

The formula $V = CR^{\frac{1}{2}}S^{\frac{1}{2}}$ (Chezy formula) appears to have as funda mental basis that if scalar relationship is maintained constant between th roughness of surface and the main dimensions, C will be a constant and th formula will apply to any size of pipe. The difficulty in using the formul in engineering practice arises from the fact that when a given material o construction is used the roughness of the surface does not alter with increas of size, and scalar relationship is not maintained. The result in practice that as the size is increased, C requires to increase also, and become dependent on some relation between roughness and diameter. Variou formulas have been proposed for arriving at C, with the result that th Chezy formula, originally of fundamentally simple form, becomes com plicated in application.

A Paper by Dr. A. Strickler[1] came to the Author's notice in 1927, and

[1] "Formulas for Velocity and Coefficient of Roughness for Rivers, Canals an Closed Conduits." Report of the Amt für Wasserwirtschaft of Switzerland, 1923.

careful perusal convinced him that the methods of analysis applied and the conclusions reached were sound and that application of the formulas recommended would lead to simplification of the calculations in flow-problems arising in engineering practice, whether on the scale of domestic water-supply and sewerage or on the much bigger scale required for large water-power installations. An important part of the investigation related to the determination of the variation of velocity with the hydraulic mean radius R when the hydraulic gradient and the roughness are constant, and for a wide range of conditions it was found that $V \propto R^{0\cdot67}$, the variation of the exponent of R being from 0·66 to 0·68, and the value being most frequently 0·67. Only for one set of observations was an exponent of 0·65 determined. The analysis confirmed also that V varied as $S^{\frac{1}{2}}$, the exponent being practically constant for all conditions. The foundation-formula recommended by Dr. Strickler with application to rivers, canals and closed conduits is:

$$V_q = KR^{\frac{2}{3}}S^{\frac{1}{2}}$$

where V_q denotes the mean velocity for quantity of flow;
R denotes the hydraulic mean radius;
S denotes the loss of head per unit length or hydraulic gradient;
and K is a coefficient depending on the roughness of the surface, and for any definite degree of roughness K is a constant.

Within close limits K is found to be equal to $\frac{1}{n}$, where n is Kutter's coefficient for roughness, and the units are metres and seconds. As Kutter's coefficients are familiar to many engineers it will be convenient to transpose the formula for metre-second units into

$$V = \frac{1}{n} R^{\frac{2}{3}}S^{\frac{1}{2}} \quad \ldots \ldots \ldots \quad (1)$$

and for foot-second units into

$$V = \frac{1\cdot486}{n} R^{\frac{2}{3}}S^{\frac{1}{2}} \quad \ldots \ldots \ldots \quad (2)$$

When it is desired to find friction-loss or hydraulic gradient instead of velocity the formulas become, for metre-second units

$$S = \frac{V^2 n^2}{R^{1\cdot333}} \quad \ldots \ldots \ldots \quad (3)$$

and for foot-second units

$$S = \frac{V^2 n^2}{2\cdot2R^{1\cdot333}} \quad \ldots \ldots \ldots \quad (4)$$

The formulas (2) and (4) are adopted by the Author for foot-second

calculations and formulas (1) and (3) for metric calculations, and ha been tested and found reliable over a wide range.

It will be understood that n is not exactly Kutter's coefficient, but is similar coefficient applicable to the above formulas.

Values of the roughness-coefficient n for use in the formulas are inc cated in Table IX in the Appendix (p. 489), for a wide range of condition These are based on the information contained in the publication referre to, supplemented by data compiled by the Author for large closed pipe conduits and tunnels up to 20 feet in diameter for which the formula found to be reliable.

As coefficients of friction loss are still frequently investigated in terms $\frac{V^2}{R}$ on the Chezy basis, the recommended formula, $S = \frac{V^2 n^2}{R^{1 \cdot 33}}$ (metre-secon units), may be transposed into the form

$$S = \frac{n^2}{R^{0 \cdot 33}} \frac{V^2}{R},$$

or

$$S = \left(\frac{n^6}{R}\right)^{0 \cdot 33} \frac{V^2}{R}.$$

The term $\left(\frac{n^6}{R}\right)^{0 \cdot 33}$ would then represent the true Chezy coefficient, whic is seen to vary both with roughness and with hydraulic radius.

It appears that $\frac{n^6}{R}$ is a factor of similarity correlating the magnitud of the roughness to the dimensions of the pipe, and that $\frac{n^6}{R}$ is proportiona to $\frac{k}{D}$, where k is a dimension representing the averaged magnitude of th roughness, and D (equal to $4R$) denotes the diameter of the pipe. Th formula may therefore, for purposes of comparison with the results c certain experiments, be written as

$$S = C_1 \left(\frac{k}{D}\right)^{0 \cdot 33} \frac{V^2}{R}, \quad \text{where } C_1 \text{ is now a constant.}$$

The experiments referred to are the very elaborate and careful serie carried out by Nikuradse with artificially prepared sand-grain surface in pipes of about 1-inch, 2-inch and 4-inch diameter, giving $\frac{k}{D}$ ratios of from $\frac{1}{30}$ to $\frac{1}{1000}$. The experiments traversed the range from viscous flow ($S \propto V$) through an intermediate stage ($S \propto V^{1 \cdot 75}$) to the stage of completely turbulen flow ($S \propto V^2$), the effect of the roughness being completely developed in th

latter stage. The results of the experiments have been published elsewhere[1].

Analysis of the results for those sections of the investigation in which roughness was fully developed, as indicated by the resistance varying as V^2, confirms that the friction coefficient on the Chezy basis varies within very close limits as $\left(\frac{k}{D}\right)^{0\cdot33}$, thus confirming the fundamental correctness for engineering calculations and the wide range of applicability of the formula $S = \frac{V^2n^2}{R^{1\cdot33}}$.

The lower limit of size of pipe for which the formula is generally suitable may be taken as 6 inches in diameter, but the limit varies with roughness, velocity and temperature of the water.

The adopted formula $S = \frac{V^2n^2}{2\cdot2R^{1\cdot33}}$ is convenient for calculation by slide-rule or logarithmic tables. It is seen that the loss of head is proportional directly to the square of the velocity and the square of the roughness-coefficient, and inversely to the one-and-one-third power of the hydraulic mean radius R. To get an appreciation of the scale of the variations, the loss in a planed wood-stave pipe having $n = 0\cdot011$ may be compared with that in a heavy lap-riveted pipe having $n = 0\cdot0155$. The ratio of losses would be $\left(\frac{0\cdot0155}{0\cdot011}\right)^2 = 2$, so that for the same size and flow the loss of head per foot in a riveted pipe might be twice as much as in a wood-stave pipe.

In order to visualize the effect of variation of R, the loss in a 4-inch diameter model with roughness fully developed may be compared with that in a 9-foot full-size conduit, all dimensions, including R, being increased 27 times. For the same values of V and n the loss of head per foot in the large conduit will be diminished to $\frac{1}{27^{1\cdot333}}$, or $\frac{1}{81}$ of that in the small conduit. If in maintenance of scalar relationship the unit of length is taken equal to one diameter and the lengths have the same number of diameters in each case, the loss per unit and the total loss in the large conduit (27 times as long as the small conduit) becomes one-third of that in the small. If therefore the loss of head in the closed passage of a siphon, or part of the loss, follows the law of pipe-flow, the results obtained from a model will not be directly applicable to a full-size apparatus, as large allowance must be made for diminution of loss with increase of size. An endeavour will be made to show later that the formula given is valid for the purpose of evaluating the variation of loss arising from change of scale in bends and in siphon-flow.

---

[1] J. Nikuradse, "Strömungsgesetze in Rauhen Rohren." Verein deutscher Ingenieure, Forschungsheft 361. Berlin 1933.

The formulas (1), (2), (3) and (4) are applicable in general to pip canals, tunnels, and open and closed conduits of regular form and of sizes, provided that the hydraulic radius is not less than 1·5 inch, and t pipe or conduit is not of the very smooth class. The formula is also appl able to rivers provided that there is sufficient regularity on a long length

Distribution of Velocity in Pipes as affected by Diameter and Nature of Surface†.

In all steady pipe-flow there is a variation of velocity across a diamet the velocity at the sides being less than the mean and the velocity at t centre greater than the mean. Where the effect of the roughness-ra is fully developed, the variation of velocity becomes greater as the roug ness-ratio D/k decreases, and this result is also found in numerous curv of velocity obtained for large pipes in connexion with the efficiency tests water-power stations.

Investigations of a number of curves for large pipes having large valu of D/k (very smooth class) indicates that steady flow is seldom found such pipes, there being either a central area in which the velocity fluctuating but the mean velocity is nearly uniform, or a steady b eccentric flow, indicating a spiral motion down the pipe. The indicatio are that in large pipes relatively much longer lengths of pipe (great numbers of diameters) are required to develop steady symmetrical fl than is the case with the small pipes of laboratory-tests, and this is cons tent with deductions which may be made from the pipe-flow formulas.

Kinetic Energy of a Flowing Stream having Non-Uniform Velocity†.

In a stream flowing with uniform velocity at all points, the kinet energy head of each filament and of the whole stream is $\frac{V^2}{2g}$. In ordina pipe-flow the velocity is not uniform, the velocity at some parts bei greater, and at other parts being less than, V_q, where V_q denotes the val of the mean velocity. The energy-head for the whole stream arrived by integrating the energies of the various filaments is always greater tha $\frac{V_q^2}{2g}$, and this increase of kinetic energy over $\frac{V_q^2}{2g}$ becomes material in t investigation of loss of head. Further, if increased kinetic energy induced by other means than the roughness of the surface, such increa will be accompanied by increase in the rate of loss of head. This is t case with flow around bends. The increase of kinetic energy in straig

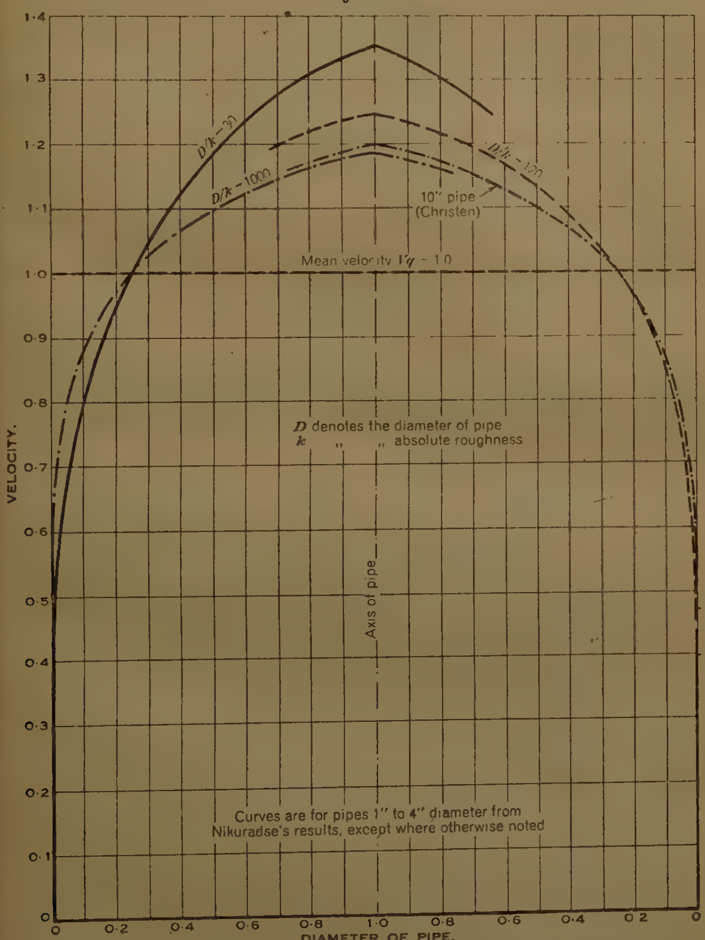
† Abridged. The MS. may be seen in the Institution Library.—Sec. Inst. C.

ipes for various roughness-ratios is indicated in Fig. 4, Plate 1, and in Table I, p. 460.

Conditions of Similarity of Flow.

Typical curves for steady flow with roughness fully developed, as determined by Nikuradse in small pipes, are shown in *Fig. 1* for D/k

Fig. 1.



Forms of Velocity-Curves in Relation to the Roughness-Ratio D/k.

ratios of 30, 120 and 1,000 respectively. It will be noted that in no case s the curve at the axis rounded off in parabolic form, all of the curves being

of pointed form, the intersection becoming flatter as the ratio D/k becom[es] greater. The forms of the curves are consistent with the overlapping [of] the friction drag-effect from each side and with diminished interaction [as] the ratio D/k increases.

Important features of the curves are the following :—

(a) The form of the curve is dictated by the ratio of diameter [to] roughness (D/k).

(b) The ratio of maximum velocity (V_m) at the axis to the mea[n] velocity (V_q) increases with decrease of the ratio D/k.

(c) The ratio of the velocity at the side (V_s) to the mean veloci[ty] (V_q) decreases with decrease of the ratio D/k.

(d) The kinetic energy of the stream increases with decrease of D/k.

(e) Variation of the quantity flowing does not affect the form of th[e] curve in relation to the mean velocity (V_q).

(f) The mean velocity for quantity occurs at practically a consta[nt] position in the pipe, that is, at about $0{\cdot}125D$ from the side[s].

Within the limits of the experiments it is clearly indicated that th[e] ratio of diameter to roughness, D/k, determines the form of the velocit[y] curve, and that similarity of flow will occur at the same values of D/k. The velocity-curve for a 10-inch pipe, determined by Christen, show[s] that the principle will apply beyond the range of the small pipes [in] Nikuradse's experiments, provided steady flow is attained with roughne[ss] fully developed.

Velocity-curves in large pipes do not always show the typical form f[or] fully developed roughness. In large pipes the ratio D/k is generally muc[h] larger than in small pipes, friction-effect is relatively much smaller, and [a] longer length of travel in straight pipe is required before steady conditio[ns] are reached. Velocity-curves for various large pipes are shown in Figs. [2,] Plate 1, and call for some remark. Fig. 2 (A) shows the curves taken on tw[o] diameters at right angles to each other in an 8-foot 9-inch diameter smoot[h] pipe in the Galloway Water Power Scheme. A contour-diagram of th[e] velocities is shown in Fig. 3, Plate 1, and is typical, not of axial flow, bu[t] of spiral flow. The readings were taken at a point on straight pipe abou[t] 34 diameters below a vertical bend. The coefficient n for the pipe is fro[m] about 0·011 to 0·0115, and ratio D/k of the order of from 3,500 to 4,00[0]. This ratio appears to be about the limit for obtaining steady axi[al] flow.

The curve in Fig. 2 (B) is for a 6-foot 8-inch diameter riveted pipe of lig[ht] construction taken a short distance below a bellmouth-inlet. A bellmout[h] inlet produces a nearly uniform entering-velocity and there has not bee[n] sufficient distance of travel to develop the full effect of the roughness.

The curve in Fig. 2 (C) for a 6-foot diameter cast-iron pipe indicat[es] symmetrical flow, but that there may not have been sufficient length [of] travel to develop fully the effect of roughness.

The curve in Fig. 2 (D) indicates symmetrical flow with almost fully developed effect of roughness.

The curve in Fig. 2 (E) indicates approach to full development of the roughness-effect but with some variation from axial flow.

Of the various curves, only those shown in Figs. 2 (D) and 2 (C) appear to represent flow with roughness approximately fully developed, and could be compared with the flow in small rough pipes.

Fig. 4, Plate 1, shows a chart plotted to indicate approximate similarity in pipes flowing with the roughness-effect fully developed, the basis being that the same value of D/k will give approximately the same form of curve. The horizontal lines of the diagram represent a range of values of D/k from 30 to 4,000. Each value of D/k corresponds to a value of $\frac{V_m}{V_q}$, the ratio of maximum to mean velocity, and a scale for $\frac{V_m}{V_q}$ is given at the left of the diagram. The smaller the value of D/k the larger is the ratio $\frac{V_m}{V_q}$, that is, the variation of velocity increases with increase in the relative roughness. Scales are also given for $\frac{V_e^2}{V_q^2}$, which indicates the increase in kinetic energy, the increase also being greater as the relative roughness increases, and for $\frac{V_s^2}{V_q^2}$, which indicates the ratio of the kinetic energy at the side of the pipe to the kinetic energy represented by the mean velocity V_q. The value $\frac{V_s^2}{V_q^2}$ becomes smaller as the relative roughness increases. The curves are plotted on the basis that absolute roughness is proportional to n^6. Three curves are given at the left of the diagram for 1-inch, 2-inch, and 4-inch diameter pipes. Points determined from Nikuradse's experiments are shown, and the agreement of the curves with the plotted points is very close. Points determined from the velocity-curves shown in Figs. 2 (D) and 2 (E), Plate 1, are indicated at * and at ‡. These two plotted points give good confirmation of the similarity relationship for pipes without internal projections. Point † indicates the position for the Humber Arm riveted pipe in respect of n and D/k. The form of the velocity-curve and the excess kinetic energy is represented by position §. The discrepancy in this case may be attributed to lack of similarity, as intermittent projecting plate-edges and rivets are not similar to a uniformly distributed roughness. It would appear that for lap-riveted pipes the form of the velocity-curve is dictated by the magnitude of the projecting plate-edges and rivets, whereas the value of n for determining loss of head is a mean between that corresponding to the general roughness of the pipe-surface and that of the plate-projections and rivets. Additional data for both flush pipes and lap-riveted pipes would be useful, but it is necessary that the

form of the velocity-curve should be ascertained along with the value of f and the combination is not very often available.

The line for the 4-inch pipe represents the conditions in a size oft used for models. Comparison shows that to obtain similarity in such model with the conditions in pipes † and ‡, the model would require to very smooth, with a value of n of about 0·0085.

Table I gives an indication of the maximum velocity, the kinet energy, and the variation of pressure at the side, in a pipe flowing with full developed roughness and symmetrical about the axis.

TABLE I.

Roughness-ratio, D/k (1)	Maximum velocity-ratio, V_m/V_q. (2)	Excess of side pressure over mean pressure. (3)	Kinetic energy. (4)
30	1·35	$0{\cdot}9 \frac{V_q^2}{2g}$	$1{\cdot}14 \frac{V_q^2}{2g}$
60	1·29	0·9 ,,	1·10 ,,
120	1·245	0·8 ,,	1·085 ,,
250	1·22	0·8 ,,	1·07 ,,
500	1·20	0·8 ,,	1·06 ,,
1,000	1·185	0·7 ,,	1·05 ,,
2,000	1·17	0·7 ,,	1·04 ,,
4,000	1·16	0·6 ,,	1·03 ,,

Note.—

D denotes the diameter of pipe;
k denotes the absolute roughness;
V_m denotes the maximum velocity (at the centre of the pipe);
V_q denotes the mean velocity for quantity.

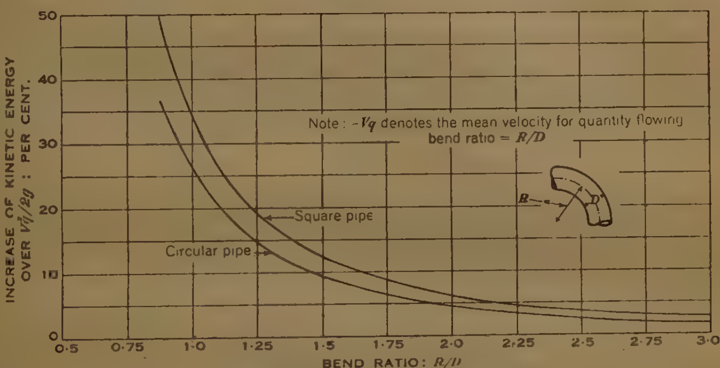
From columns (3) and (4) of the Table it is seen that the pressure rea at the side of a pipe is always greater than the mean pressure, and the kinet energy of the stream is always greater than $\frac{V_q^2}{2g}$. The point is of importanc in any problem in which the energy-head $\frac{V_q^2}{2g}$ becomes appreciable i relation to the pressure-heads, and is worthy of careful consideration in th analysis of turbine-efficiency tests.

MOTION AROUND BENDS.

When water is constrained to flow in a curving path around a bend becomes subject to centrifugal force. Restraining forces directed toward the centre of the curve are required to keep the water in its curving path and the net result in the case of a bend in a horizontal plane is increase o pressure on the outer parts of the bend accompanied by decrease o

velocity, decrease of pressure on the inner parts accompanied by an increase of velocity, a lowering of the mean pressure-head and an increase in the mean kinetic energy. The difference in pressure between the outside surface and the inside surface counteracts the centrifugal force, and the whole effect tends towards the production of free vortex-flow. In fully developed vortex-flow, VR is constant, V denoting the velocity of a filament and R its radius of curvature. If in a sharp bend the inner radius is 4 units and the outer radius 12 units, the theoretical vortex-velocities at the inside and outside respectively would be as 12 is to 4. A much greater variation of velocity may therefore be expected at a cross section at a bend than occurs normally in a straight pipe, and consequently also a much greater increase of kinetic energy. The increase of kinetic energy, on the assumption of theoretical vortex-flow, has been evaluated for bends of

Fig. 5.



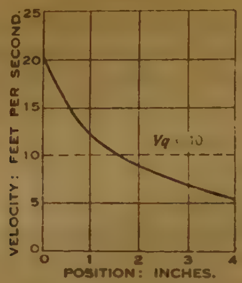
THEORETICAL VORTEX-FLOW.

Curves showing increase of Kinetic Energy in Relation to Sharpness of Bend.

square and circular section having ratios of R/D from 3·0 to 0·875 (R denoting the radius of the axis of the bend and D the diameter of the pipe), and is shown by curves in *Fig. 5.* The increase is quite small for a ratio 3 which denotes an easy bend, but mounts rapidly for sharper bends and rises to 50 per cent. for a square pipe with a ratio of 0·875. The rapid rise in kinetic energy with sharpness of bend is accompanied in somewhat similar fashion by a rapid rise in the rate of loss of head, as will be seen later, the effects being of a similar nature to those found in straight pipes, but on a larger scale.

Taking a 4-inch square pipe with mean velocity $V_q = 10$, the theoretical curve of velocities across the pipe for vortex-flow is shown in *Fig. 6* (p. 462), for $R/D = 0{\cdot}875$. For this case $V_e^2 = \dfrac{\Sigma Q V^2}{\Sigma Q} = 150$, bringing out a 50 per cent. increase of kinetic energy over that due to uniform flow in a

Fig. 6.

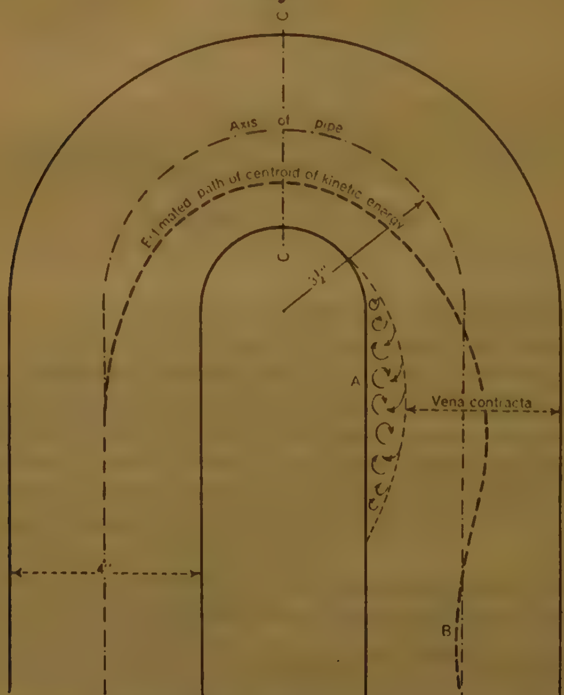


SECTION CC in *Fig. 7.*

Curve of Velocity for Vortex-Flow.

straight frictionless pipe, and that without any increase in the quantity flowing. Almost half the total kinetic energy passes in the 4-inch by ½-inch strip next the inner wall. The centroid of the stream of kinetic energy is at about 0·9 inch from the inner wall, as compared with 2 inches in the straight pipe. The path of the centroid cannot change suddenly, and for the sharp bend of the example with a 180-degree angle may be approximately as indicated in *Fig. 7.* The further the path deviates from the axis the greater

Fig. 7.



APPROXIMATE PATH OF CENTROID OF ENERGY FOR SHARP BEND, 4-INCH BY 4-INCH SECTION ($R/D = 0{\cdot}875$).

is the kinetic energy of the whole stream, and so also the rate of loss of head may be expected to be greater. The head to produce this increase of kinetic energy is abstracted from the pressure-head, but this transfer of energy from one state to another does not represent a loss of energy, and the fall in pressure-head must not be attributed to loss by friction. A transition-path is necessary at the inlet-end of the bend, and this is shown commencing in advance of the bend, which is in accordance with the indications of experiment. The path reaches maximum deviation near the end of the first quadrant and remains parallel to the axis for some distance, but begins to deviate towards the axis before the end of the second quadrant, and then follows a transition-path back to the centre of the stream in the outlet-leg. The transition-path is shown crossing the axis slightly and coming back to the straight on a reverse curve, this being in accordance with the pressure results in certain small-scale experiments. It is probable that a swinging motion or a spiral motion would continue for a considerable distance in large pipes.

When the high-velocity jet following the inside of the bend begins to lose the effect of the restraint of the curving walls, which it does to some extent in advance of the outlet-end of the bend, it tends to continue onwards with less curvature and to leave the inner wall. A vena contracta develops with the main flow confined to less than the full width of the pipe, the remainder of the space indicated at A, *Fig. 7*, being occupied by turbulent water with little forward velocity. An actual increase in the mean velocity is now produced due to the diminished passage-space. Redistribution of velocities and pressures begins in the converging part of the vena contracta and continues beyond it, and normal conditions of straight pipe-flow are attained at some distance beyond the end of the bend. In the course of the redistribution a substantial part of the excess kinetic energy is reconverted into pressure-energy, and the rise in pressure need not be attributed to any mysterious gain of total energy.

The progressive loss of head around such a bend is extremely difficult to determine. Attempts have been made to arrive at the losses, by ascertaining the loss of pressure at a number of sections by means of gauges at the sides of the passage and accepting the mean recorded loss of pressure as a measure of the loss of head. The Author has pointed out elsewhere that a major source of error arises from the fact that a large part of the recorded loss of pressure is balanced by increase of kinetic energy, and to that extent the loss of pressure does not represent actual loss of energy-head. A further source of error arises from the fact that pressures recorded at the walls do not represent pressures in the actual stream, and until methods are found for correctly ascertaining pressures at a sufficient number of points in a series of cross sections distributed throughout the stream, accurate analysis will not be possible. Sufficient analysis has, however, been done on certain experiments to show that loss of energy-head is progressive, and that increase of kinetic energy in the stream,

from whatever source arising (roughness of the walls, vortex-flow, or vena contracta), is accompanied by increased rate of loss of the total energy head. From incorrect analysis of pressure-readings various conclusions have been reached as to the distribution of the loss, such as that the loss is confined to the outlet-end of the bend, or that it occurs partly at the inlet and partly at the outlet. The fact must be that extra loss commences as soon as the centroid of the energy-stream leaves the axis of the pipe, increases with increasing deviation, and decreases beyond the vena contracta, coming back to normal at some distance beyond the outlet-end of the bend. It may be that in sharp bends the rate of loss is greatest at or near the vena contracta. From the best analysis possible with the rather inadequate data available it appears probable that in a 180-degree bend of sharp radius, one-third of the loss caused by the bend takes place in advance of the 90-degree point, one-third between 90-degree and 180-degree points, and the remaining third in a length of 2 or 3 diameters beyond the end of the bend.

Losses in bends have hitherto generally been expressed in the form $K \cdot \frac{V^2}{2g}$, V denoting the mean velocity for quantity and K being a coefficient, taken as constant. The Author has been convinced for some time that this method of expressing the loss is misleading, in that the coefficient, K, must vary with the scale, it being found from experience that much smaller losses occur in large bends than in small bends of similar form and surface conditions. It appears that the practical method of assessing bend-losses will be (*a*) to find the extra loss in the bend over and above the loss that would occur in the same length of pipe if straight, and (*b*) then to determine the extra length of straight pipe which would account for the extra loss. This extra length should be expressed as a certain number of diameters of the pipe and it will then be applicable for reckoning the loss whatever be the diameter of the pipe, the loss being calculated by the pipe-flow formula. It remains to apply checks to see whether the method agrees with the results of such experiments as are available on models constructed of similar form and with the same coefficient of roughness on different scales. If the pipe-flow formula applies to bend losses, then with variation of scale (but with n remaining constant), the loss per linear foot will vary as $\frac{1}{D^{1\cdot333}}$ and the loss per unit of diameter will vary as $\frac{1}{D^{0\cdot333}}$. Thus, if the diameter of the passage or scale of the model is increased 4 times, the loss per unit for the same velocity should be reduced to 0·63.

If the small model is so small and the velocities so low as to introduce additional resistance due to viscosity, then the decrease of resistance from small to large should be more than the figure just brought out. That is, for a 4-times increase of scale, the resistance in the large model should be less than 0·63 of its value in the small model.

Professor Gibson and Messrs. Aspey and Tattersall give a Table of results [1] of comparative tests on two siphon-models, one with a 3-inch by 1-inch passage, and the other with a 12·125-inch by 4-inch passage. The hydraulic radius R for the small model is 0·375 inch and for the large 1·5 inch. The small model is therefore within the range in which viscosity has some effect in increasing the frictional resistance on straight pipes with low velocities.

The mean velocity in the small siphon-model is, however, not. ow, and vortex-conditions in the sharp bends lead to high local velocities and turbulence and about seven-fold increase of resistance as compared with a straight pipe, and it may be surmized at once that the flow-conditions in both the small and the large models will be mainly turbulent.

The large models were found to have much greater coefficients of discharge than the small models, and analysis may be made to see whether the results are consistent with the scalar effect in reducing friction as found in ordinary turbulent pipe-flow, assuming that the roughness of surface is the same in both models. It is assumed that the head on the large siphon is increased 4 times to correspond with the increase of scale.

The following results were found or deduced for the small model in the second example in the Table referred to.

Head	0·75 foot
Friction-loss	0·51 ,,
Discharge-head at outlet	0·24 ,,
Coefficient of discharge on throat-velocity . .	0·910
,, ,, outlet velocity. .	0·565

If x denotes the ratio of the velocity in the large model to the velocity in the small model, then for the large model, bearing in mind that friction-loss should be reduced to 0·63 of its former value :

$$\begin{aligned} \text{Friction-loss} &= 0{\cdot}51 \times 0{\cdot}63 \times x^2 = 0{\cdot}32x^2. \\ \text{Discharge-head} &= \phantom{0{\cdot}51 \times 0{\cdot}63 \times x^2 = {}} 0{\cdot}24x^2. \\ \text{Total head} &= 0{\cdot}56x^2, \\ &= 4 \times 0{\cdot}75 = 3{\cdot}0. \end{aligned}$$

Therefore $$x^2 = \frac{3{\cdot}0}{0{\cdot}56} = 5{\cdot}36 \text{ and } x = 2{\cdot}31.$$

The ratio of discharge-head to total head becomes $\frac{0{\cdot}24}{0{\cdot}56} = 0{\cdot}43$ and the coefficient of discharge, that is, the ratio of the discharge-velocity to the theoretical spouting-velocity, would be $\sqrt{0{\cdot}43} = 0{\cdot}655$.

[1] Footnote (²), p. 451.

The ratio of the coefficients of discharge in the large and small models is therefore, by calculation, for four-fold increase of scale,

$$\frac{0{\cdot}655}{0{\cdot}565} = 1{\cdot}16,$$

or an increase of 16 per cent. in the large model as compared with the small. The increase found in the experiment was 16 per cent.

The result is consistent with the flow being essentially turbulent and the ordinary pipe-flow formula being applicable for change of scale.

Similar reasonably close correspondence between calculated and experimental results is found for the other examples in the Table.

Two cases of comparative experiments with models of different scale carried out in Italy are referred to by Professor Gibson*. These are for the Carron and Camuzzoni siphons respectively, and the smaller models were much larger than Professor Gibson's small model, so that viscosity effects could be neglected. The change in coefficients from 0·512 to 0·54 in the one case and from 0·50 to 0·53 in the other are again accounted for very closely by the change of scale producing reduction in the frictional resistance as calculated by the pipe-flow formula. The Camuzzoni experiment considered is the one corresponding to a head of 17·4 feet on full size. The one for a 16·4-foot head is discarded, as the enormous drop in coefficient both in the model and in full-size as compared with a 17·4-foot head would indicate some disturbing factor, such as admission of air, which caused departure from usual hydraulic flow, and reduced the efficiency.

These and other checks which are available indicate that the pipe-flow formula is applicable with a practical degree of accuracy to the flow in bends and siphons in respect of change of scale.

There remains to be investigated the question whether or not the coefficient of roughness has the same relative effect in flow around bends and in siphons as it has in straight-pipe flow.

Some indirect evidence is available which indicates that losses in respect of surface-roughness should follow the pipe-flow formula. Very direct confirmation is fortunately furnished by Messrs. Davies and Puranik on p. 87 of a Paper by them, published in 1936†. The teakwood therein described offered smaller resistance when new and before thorough soaking than after thorough soaking, the values of n as worked out by the Author from the experimental results of the friction loss in square pipes being 0·0075 and 0·0084 respectively.

$$\text{The friction-loss ratio} = \left(\frac{0{\cdot}0084}{0{\cdot}0075}\right)^2 = 1{\cdot}26.$$

That is, the thoroughly soaked wood surface should offer 26 per cent. more resistance than the same surface when first used.

* Footnote (2), p. 451.

† Footnote (4), p. 451.

Mr. Davies found in one case that the new wood surface gave a resistance of $0{\cdot}26\frac{V^2}{2g}$ in a bend, whereas after thorough wetting the resistance was $0{\cdot}33\frac{V^2}{2g}$, or an increase of 27 per cent., which compares with the figure of 26 per cent. obtained above from the roughness-coefficients. This case affords satisfactory confirmation of the principle, and as Mr. Davies also ascertained that the losses in the pipes and bends varied as V^2, it may therefore be accepted that the losses in bends and siphons follow the law

$$S \propto \frac{V^2 n^2}{R^{1\cdot33}}.$$

It remains to have the losses in bends expressed in a form which will enable the formula to be applied. It is obvious that to state the bend-loss as $K\frac{V^2}{2g}$ will not be appropriate, as K has now been found not to be constant, but to vary both with the scale and with the coefficient of roughness, and much confusion has resulted from the general but unwarranted assumption that K is a constant. The only appropriate method will be to convert the loss in a bend, as ascertained by experiment, into the loss in a length of straight pipe of the same smoothness equal to a certain number of diameters of the pipe. If from the total loss expressed as a number of diameters, the length around the bend (also expressed in diameters), to which the loss applies, is deducted, the remainder will be the extra loss arising from the introduction of the bend.

Mr. Davies in his recent Paper [1] has given, in a number of Tables, the coefficients which he has ascertained for the loss in 4-inch by 4-inch square-section teakwood bends, and has also given data for the losses in similar straight square pipes. The necessary information is therefore available, probably for the first time, for converting the losses in a series of bends into an equivalent number of diameters of straight pipe, and this has been done by the Author for the more representative cases, the results being given in Table II (p. 468). The coefficients taken for the calculations are the mean of the two highest results out of four or more observations at different velocities, the purpose being to obtain covering figures for the losses suitable for use in hydraulic calculations. The results for the extra losses in the bends have been plotted in graphical form in *Fig. 8* (p. 468).

The conclusion is reached that the losses shown by the lines on *Fig. 8* may confidently be used in conjunction with the pipe-flow formula for calculating the losses in bends of square-section conduits. The procedure would be to take the whole length of the conduit and to add to it the extra lengths for the several bends as obtained from *Fig. 8*, and to treat the

[1] Footnote (4), p. 451.

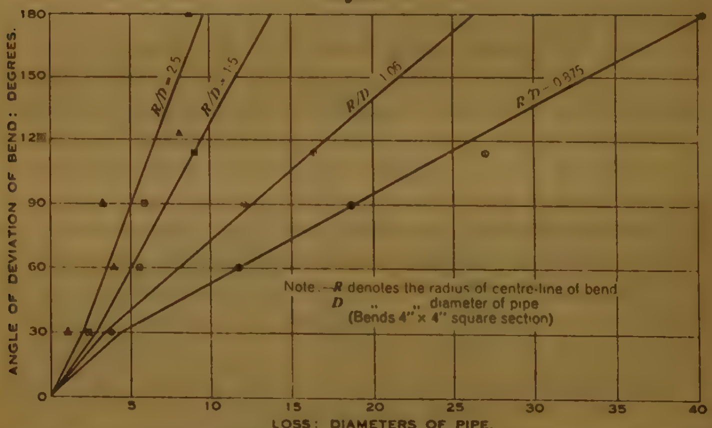
TABLE II.—BEND LOSSES IN 4-INCH BY 4-INCH SQUARE TEAKWOOD BENDS.

Loss per diameter in straight pipe $= 0{\cdot}0187 \frac{V^2}{2g}$ feet.

$n = 0{\cdot}0084$.

Bend R/D.	Item.	Angle of deviation: 180°	122° 30′	114° 15′	90°	60°	30°
0·875	Coefficient . . .	0·86	—	0·595	0·435	0·295	0·13[illegible]
	Loss: No. of diam.	46·0	—	31·8	23·2	15·8	7·3
	Length taken: diam.	5·7	—	4·7	4·4	4·0	3·5
	Extra loss in bend .	40·3	—	27·1	18·8	11·0	3·8
1·06	Coefficient . . .	—	—	0·405	0·315	—	—
	Loss: No. of diam.	—	—	21·6	16·-8	—	—
	Length taken: diam.	—	—	5·2	4·7	—	—
	Extra loss in bend .	—	—	16·4	12·1	—	—
1·5	Coefficient . . .	—	—	0·28	0·205	0·19	0·115
	Loss: No. of diam.	—	—	15·0	11·0	10·0	6·1
	Length taken: diam.	—	—	6·0	5·3	4·5	3·8
	Extra loss in bend .	—	—	9·0	5·8	5·5	2·3
2·5	Coefficient . . .	0·365	0·305	—	0·19	0·18	0·10
	Loss: No. of diam.	19·6	16·3	—	10·2	9·6	5·4
	Length taken: diam.	10·9	8·2	—	6·9	5·6	4·3
	Extra loss in bend .	8·7	8·1	—	3·3	4·0	1·1

Fig. 8.



EXTRA LOSS DUE TO BEND EXPRESSED AS NUMBERS OF DIAMETERS OF STRAIGHT PIPE[1].

increased length so obtained as the equivalent total length of the condui for purposes of calculation. For example, take a square concrete conduit

[1] Based on the results of experiments by P. Davies (Journal Inst. C.E., vol. 1935–36), p. 83. (February 1936)).

feet by 5 feet, flowing full at 10 feet per second, having a length of 500 feet and two bends, (1) a 60-degree angle and $R/D = 1{\cdot}5$, and (2) a 90-degree angle and $R/D = 0{\cdot}875$.

Length of conduit	= 500	feet.
Extra for bend (1) 5 diameters × 5 feet	= 25	,,
,, ,, ,, (2) 19 diameters × 5 feet	= 95	,,
Total equivalent length . .	= 620	,,

If n for the concrete surface is 0·013, the loss S per foot would be :

$$S = \frac{V^2 n^2}{2{\cdot}2 R^{\frac{4}{3}}},$$

where $$V = 10,\ n = 0{\cdot}013 \text{ and } R = \frac{5}{4} = 1{\cdot}25,$$

from which $S = 0{\cdot}0057$ (foot loss of head per foot), and the total loss including bends $= 620 \times 0{\cdot}0057 = 3{\cdot}5$ feet, the bends accounting for about one-fifth or 0·7 foot.

Reliable data for the losses in bends of circular pipe-form with a determined degree of smoothness of surface appear to be lacking. It has been pointed out, however, that theoretical vortex-motion in a circular pipe gives rise to less increase of kinetic energy than corresponding motion in a square pipe and this may indicate rather smaller losses, but careful experiment would be required to ascertain the facts. It is considered, however, that the graphs of *Fig. 8* may be used also for circular pipes, for the purpose of arriving at conservative discharging capacity.

A check has been made to see whether the bend-losses are applicable in the combinations as used in siphons.

Mr. Davies in Table IV of his Paper [1] gives particulars of a test on an unvarnished teakwood model-siphon of 12-inch by 4-inch section, the principal results being :

Head producing flow : feet.	$\sqrt{2gH}$	V_g, mean velocity at throat.	C, coefficient of discharge.	V_o, outlet velocity.	$\frac{V_o^2}{2g}$: foot.	Friction-head : feet.
2·895	13·65	12·12	$\frac{12{\cdot}12}{13{\cdot}65} = 0{\cdot}884$	7·48	0·873	2·022

The figures in the last three columns have been added by the Author

[1] Footnote ([3]), p. 451.

to arrive at the head lost in friction (2·022), this being the differen between the total head (2·895 feet) and the kinetic head of the outflowi water (0·873 foot).

The total length of passage is 16 diameters, the increased length fou by adding the appropriate extra for each bend is 50 diameters, the asce tained loss for 1 diameter of 4-inch by 4-inch straight pipe for $V_q = 12$· is 0·043 feet, and the total calculated loss therefore is $50 \times 0{\cdot}043 = 2$· feet, which agrees closely with the loss of 2·022 feet obtained from t experiment. There is, therefore, confirmation that the experiment results obtained with bends are applicable also to the bends in sipho provided the surface-roughness is the same. The method of calculati adopted is indicated in the example given on pp. 477–478.

Consideration of the rates at which friction-losses increase on the o hand in respect of increase of n, the coefficient of roughness, and decrea on the other hand in respect of increase in the scale, will indicate that if surface of definite smoothness and fixed value of n, say N_m, is available f models and N_m is low (but not too low) as compared with the coefficien for practical surfaces, it will be possible to choose a scale so that the mod will give the same coefficient of discharge as the full size.

If N_a is the coefficient of roughness for the surface in the actual full-si apparatus and N_m is the coefficient for the surface in the model, then th increase of friction is in the ratio $\left(\frac{N_a}{N_m}\right)^2$.

If $\frac{1}{A}$ is the scale-ratio of model to actual, then the decrease of friction head from model to actual is in the ratio $\frac{1}{A^{0{\cdot}333}}$. If the increase from th one source is equal to the decrease from the other, then the model woul give directly the true coefficient for the actual, and for this condition to b satisfied,

$$\left(\frac{N_a}{N_m}\right)^2 = A^{0{\cdot}333},$$

or

$$A = \left(\frac{N_a}{N_m}\right)^6.$$

Expressed in words this means that the ratio of diameter to absolut roughness must be the same for both.

For example, if it is desired to have a test for a full-size apparatus fo which $n = 0{\cdot}012$ by means of a model with surfaces having $n = 0{\cdot}0084$ and to obtain results which are directly applicable, the scale of the mode will be given by the sixth power of $\left(\frac{0{\cdot}0084}{0{\cdot}012}\right)$, or $\frac{1}{8{\cdot}5}$.

If the model-surface had $n = 0{\cdot}0064$ the scale of the model would be the sixth power of $\left(\frac{0{\cdot}0064}{0{\cdot}012}\right) = \frac{1}{43}$.

The former scale-ratio $\frac{1}{8{\cdot}5}$ would be a practical one for many siphon-investigations but not for all.

The latter scale-ratio would in general give impracticably small models and with A less than 43, and a model-surface having $n = 0{\cdot}0064$, the model would give higher coefficients than the actual. It is obvious, however, that it is not necessary to choose a scale-ratio in the manner indicated. The proper coefficient for the actual apparatus can be derived by simple calculation from the coefficient of the model by correct application of the relationships in the pipe-flow formula, provided that the coefficient of roughness for the model is known. It will be realized that if the results of model-tests on siphons are to be of any practical use, the coefficient of roughness of the surface employed must be ascertained, and special experiments may be required for this purpose.

Siphonic Flow.

In the usual form of siphon there is an inlet-mouth with its top below the reservoir or head-water, a passage rising up from the inlet, bending over a crest and turning down vertically or in a sloping direction to an outlet at an appropriate distance below the head-water level. The crest is generally at or near the head-water level and the lower end of the siphon is generally arranged with a bend so that the discharge is horizontal or nearly so. In a properly designed siphon the difference in head between the reservoir-level and the outlet water-level (suitably reckoned according to circumstances) is available for generating flow, and is used partly in producing the energy of the issuing water $\left(\frac{V_o^2}{2g}\right)$ and partly in overcoming the friction-losses in the passages. The water has to be lifted above the reservoir water-level to get over the crown-bend, and energy-head (represented by the height to be lifted) must be available for this purpose. In addition to the actual lift a large amount of head is required to generate the high velocities which occur in the crown-bend over the crest. The necessary head for these purposes in a siphon is furnished by the production of a partial vacuum, or a lowering of the pressure below atmospheric pressure, in a portion of the passage so that the atmospheric pressure (or part of it) on the surface of the reservoir becomes available for forcing the water up over the crest and generating its velocity. It will be recognized that the atmospheric pressure imposes strict limits on the availability of siphonic action. A siphon which was found to work satisfactorily at sea-level might give rise to trouble if copied and installed at 6,000 feet above sea-level. It is therefore necessary to give particular attention to conditions

in the crown-bend over the crest of the siphon, where maximum reduct of pressure occurs, if the limitations of siphonic flow are to be apprecia and data made available for control of design.

As happens in all regular bends the flow around the crown-bend of siphon is characterized by approximation to free-vortex flow, the theoret relationship being $VR =$ constant, where V is the velocity in a flow filament and R is its radius of curvature. Crown-bends are often of sh radius in relation to the depth of passage, and in such cases the velocity the inner side of the bend (crest of the siphon) is much larger than the m velocity, whereas the velocity at the outside of the bend is less than the m velocity. As the energy-head is proportional to $\frac{V^2}{2g}$, the energy per in a filament at the inside of the bend may be several times greater th the mean. The height, h, of any point in the crown-section above reservoir water-level represents a loss of pressure-head. The energy-he $\frac{V^2}{2g}$, imparted to the water entails an equal loss of pressure-head, and sum of these two losses is $\left(h + \frac{V^2}{2g}\right)$. There will, in addition, be some loss pressure-head as the result of energy destroyed by friction in the passa from the reservoir to the point in question. Designating this loss by f, total drop in pressure-head as compared with the pressure at the surface the reservoir (which is atmospheric pressure) will be $\left(f + h + \frac{V^2}{2g}\right)$. I points at elevations below the level of the surface of the reservoir, h w be negative.

The problem at the crown-bend is to make the design such that at points the summation $\left(f + h + \frac{V^2}{2g}\right)$ is less than the atmospheric head an adequate margin. The atmospheric-head at sea-level is equivalent about 33 feet-head of water, and decreases roughly by 1 foot for each 8 feet of elevation.

The principal factor to be taken into account at the crown is the lar variation in pressure across the width in sharp bends. The velocity at t outside of the bend is less than the mean and the pressure is consequent higher. The velocity at the inside of the bend is higher than the mean, a the pressure is consequently lower. The decrease of pressure from t axis of the passage to the inside of the bend is always greater than t increase from the axis to the outside of the bend. Taking V_q as the me velocity for the quantity discharged, the energy-head for uniform flo is $1{\cdot}00\frac{V_q^2}{2g}$. Table III gives the corresponding energy-heads at the outsi and inside of bends having various ratios of R/D, based on theoretic vortex-flow. The figures are necessarily approximate when applied

uctical cases but are considered to be much more reliable on the engineer-g scale than any results based on pressure-readings taken at the sides of e passages in small models.

TABLE III.—VELOCITY-HEADS AT OUTSIDE AND INSIDE OF BENDS.

Bend of ratio R/D:	Velocity-head in terms of $\frac{V_q^2}{2g}$:	
	Outside of bend.	Inside of bend.
2·50	0·69	1·56
2·00	0·64	1·78
1·50	0·56	2·25
1·25	0·51	2·77
1·06	0·46	3·63
1·00	0·44	4·00
0·80	0·38	7·11

For example, if in an actual case at the crown-bend of a siphon, $V_q = 24$ et per second, then $\frac{V_q^2}{2g} = 9$ and the velocity-head at the inside of the end for $R/D = 2{\cdot}5$ would be $9 \times 1{\cdot}56 = 14$ feet head; the drop in ressure, allowing for some friction-loss in the passage up to the crest, ould be rather more than this, but flow-conditions would be satisfactory s less than half of the available atmospheric pressure is used up. For $/D = 1{\cdot}25$ the velocity-head at the inside of the bend would be $9 \times 2{\cdot}77$ = 25 feet head, and flow-conditions might be expected to be marginal with me slight drop in efficiency due to gases dissolved in the water coming out f solution and expanding to considerable volume under the low pressure. 'or $R/D = 1{\cdot}00$, the velocity-head from the Table would be $9 \times 4{\cdot}00 = 36$ et-head and as this is more than the atmospheric head available the heory of vortex-flow would not apply, and the siphon, if it worked, would perate with reduced efficiency and with great tendency to erosion of urfaces, even metal surfaces, at the places of lowest pressure.

The foregoing considerations on flow and losses in pipes, bends and iphons lead to a rational method of investigation which appears to be pplicable to nearly all types of siphon having bends of regular form. The ethod was outlined by the Author in the Discussion on Mr. Davies' 'aper on the "Laws of Siphon Flow,"[1] but falls to be amended to suit he application of the pipe-flow formula by having the losses in bends xpressed as a number of diameters of the pipe instead of as a coefficient f $\frac{V^2}{2g}$. The pressures at crown-bends also require to be amended, by llowing for the appropriate variations at the outer and inner surfaces in ccordance with Table III. In illustration of the method, two examples

[1] Footnote ([3]), p. 451.

will be considered, namely, type A in which the outlet is contracted to 1 the area of the passage, and type B in which the outlet is of the sa section as the passage. The characteristic feature of type A is tha may be applied to heads considerably greater than the atmospheric he and with further contraction of the outlet it may be applied to any he whereas type B is only applicable for heads well below the atmosphe head. In the diagrammatic method used, the vertical limbs of siphc where such occur, are developed on to a slope to enable the course of va tions in losses, velocity-heads, and pressures to be followed along the wh length of the siphon.

The diagrams are based on plotting the losses and velocity-hea downwards from a datum line fixed at the height of the atmospheric he (in feet of water) above the head water-level. At sea-level this height about 33 feet; at 6,000 feet elevation, about 26 feet. By this procedure lesser availability at high elevations is made clear and can be provided f

In all cases the passage is taken as 4 feet by 4 feet, the upper bend a 180-degree angle with $R/D = 1{\cdot}06$, and the lower bend a 90-deg angle with $R/D = 1{\cdot}50$. The inlet is taken of easy form with negligi resistance, apart from the loss in the passage. n for a good smooth co crete surface is taken as 0·012.

The head for the first example will be taken as 40 feet and for t second 15 feet.

The lengths of passage required for a 40-foot head are:

Inlet-passage, equivalent to	2 diameters.
Crown-bend, plus 3 diameters	6 ,,
Straight passage	5 ,,
Bottom bend, plus 3 diameters . . .	5 ,,
Total	18 diameters.

The lengths taken for the bends include 1 extra diameter at the inle end and 2 extra diameters at outlet-end.

It is necessary in the first place to find the loss in 1 diameter of t straight passage (4-foot length) in terms of $\frac{V^2}{2g}$. The formula for loss p foot is $S = \frac{V^2 n^2}{2{\cdot}2R^{1{\cdot}333}}$, or $S = \frac{V^2}{2g} \times \frac{64{\cdot}4n^2}{2{\cdot}2R^{1{\cdot}333}} = 0{\cdot}0042\frac{V^2}{2g}$.

The loss per diameter is 4 times as much and therefore $= 0{\cdot}0168\frac{V^2}{2g}$

If any portion has velocity greater or less than V the correspondin loss will be reckoned to vary as the 2·67th power of the velocity and suitable adjustment made on the length of such portion. In Table IV, th losses are set out in terms of numbers of diameters of the 4-foot passag the numbers being adjusted where the velocity is greater or less than V.

TABLE IV.—SIPHON (TYPE A) WITH 40-FOOT HEAD.

Losses in terms of diameters and $\frac{V^2}{2g}$.

$\left(\text{Loss per diameter} = 0.0168 \frac{V^2}{2g}.\right)$

Section.	Velocity.	Length as number of diameters.	Extra length for bend, etc.	Total length for loss.	Length adjusted for velocity.	Loss of head.
Entrance passage.	$\frac{V}{2}$	2	—	2	1·0	$0{\cdot}02 \frac{V^2}{2g}$
Upper bend	V	6	26	32	32	0·54 ,,
Straight .	V	5	—	5	5·0	0·08 ,,
Lower bend	V	5	7	12	12	0·20 ,,
Taper .	av. 1·5 V	(1·5 included in lower bend.)	—	—	Extra 3·0	0·05 ,,
Discharge .	$2V$	—	—	—	—	4·50 ,, = (4×1·125)
	Totals	18	—	—	53·0	$5{\cdot}39 \frac{V^2}{2g}$

It will be seen that in this case by far the greatest part of the total head is used in creating velocity at the outlet. In arriving at the figure of $4{\cdot}50 \frac{V^2}{2g}$ for the discharge-head it is assumed that the increase in kinetic energy in the lower bend due to vortex-flow (about 12½ per cent.) is maintained through the outlet.

Equating the total loss of head $\left(5{\cdot}39 \frac{V^2}{2g}\right)$ to the available head of 40 feet,

$$\frac{V^2}{2g} = \frac{40}{5{\cdot}39} = 7{\cdot}4, \text{ and } V = 21{\cdot}8 \text{ feet per second.}$$

The discharge head $= 4{\cdot}5 \frac{V^2}{2g} = 4{\cdot}5 \times 7{\cdot}4 = 33{\cdot}3$ feet,

and 6·7 feet is left as being used in overcoming friction. The various losses for use in plotting the developed diagram shown in Fig. 9, Plate 1, are given in Table V (p. 476), the losses in the crown-bend being allocated in the proportions ⅓ up to the crest, ⅓ in the next 90 degrees and ⅓ in the two diameters beyond the bend.

At the crown of the upper bend ($R/D = 1{\cdot}06$) the velocity-head at the outside (from Table III) would be $7{\cdot}4 \times 0{\cdot}46 = 3{\cdot}4$ feet, a decrease of $9{\cdot}6 - 3{\cdot}4 = 6{\cdot}2$ feet from the head used for plotting, and at the inside it would be $7{\cdot}4 \times 3{\cdot}63 = 26{\cdot}8$, an increase of $26{\cdot}8 - 9{\cdot}6 = 17{\cdot}2$ feet over the

head used for plotting the line of loss of head. The maximum loss of h
at the crest, including friction-loss, would therefore be $11{\cdot}07 + 17{\cdot}2 = 28$
feet, which is approaching absolute vacuum, and 40 feet may therefore
taken as the practical limit of head for this type, with an outlet-area eq
to half the passage-area, when used at sea level.

The spouting-velocity for a 40-foot head would be $8\sqrt{40} = 50{\cdot}5$ f
per second. The mean velocity in the passage was calculated as 2
feet per second and would be double this amount at the outlet, or 43·6 f
per second.

TABLE V.—SIPHON (TYPE A) WITH 40-FOOT HEAD.
Losses in feet of head.

Section.	Friction-loss and discharge-loss.	Cumulative friction-loss.	Velocity-head.	Total cumulative loss of pressure head
Entrance passage . .	$6{\cdot}7 \times \frac{1}{53} = 0{\cdot}12$	0·12	0 to 7·4	7·52
Upper bend:				
1. To 90° . . .	$6{\cdot}7 \times \frac{10{\cdot}7}{53} = 1{\cdot}35$	1·47	$7{\cdot}4 \times 1{\cdot}3 = 9{\cdot}6$	11·07
2. 90° to 180° . .	,, ,, = 1·35	2·82	,, ,, = 9·6	12·42
3. 180° + 2 diam. .	,, ,, = 1·35	4·17	$7{\cdot}4 \times 1{\cdot}0 = 7{\cdot}4$	11·59
Straight	$6{\cdot}7 \times \frac{5}{53} = 0{\cdot}63$	4·80	7·4	12·20
Lower bend	$6{\cdot}7 \times \frac{12}{53} = 1{\cdot}52$	6·32	$7{\cdot}4 \times 1{\cdot}125 = 8{\cdot}3$	14·62
Taper	$6{\cdot}7 \times \frac{3}{53} = 0{\cdot}38$	6·70	8·3 to 33·3	15 to 4
Discharge	33·30	6·70	33·3	40
	Total 40·00			

The coefficient of discharge reckoned on the passage velocity is $\frac{21}{50}$
= 43 per cent., and on the outlet velocity is 86 per cent.

The discharging capacity would be $(4 \times 4 \times 21{\cdot}8) = 349$ cusecs.

From Fig. 9, Plate 1, it will be seen that the friction-loss is progressi
and is most rapid at the bends. The line denoting mean pressure-head
the other hand shows a regain at the outlet-end of the bend, which, howev
does not represent regain of total energy. It is produced by reduction
the kinetic energy. The most significant part of the diagram from t
point of view of the operation of the siphon is the line denoting pressu
head at the inside of the bend, and one of the main objects of the diagra
is to ascertain the position of this line in relation to the crest of the sipho
If the line drops to the crest, it is indicated that the whole of the availal

atmospheric pressure has been utilized and absolute vacuum is produced, so that operating conditions would not be satisfactory. In Fig. 9, Plate 1, it is seen that the line falls to about 5 feet from the crest. If it is desired to improve a condition which indicates too close an approach to vacuum, this may be done either by reducing the head—that is, raising the outlet to a higher level, which would have the effect of reducing the velocities and raising the lines of the pressure-head—or by increasing the ratio R/D of the crown-bend, the principal effect of this being to raise the line showing the pressure at the inside of the bend.

The example also illustrates the point that a siphon may be satisfactory at a low elevation but not at a high elevation. If the diagram (Fig. 9, Plate 1) had been plotted for a high elevation with the atmospheric head equal to 28 feet of water instead of 33 feet the base-line for losses and all the pressure-lines would have been lowered by 5 feet and the pressure-line at the inside of the bend would have touched the crest, thus indicating absolute vacuum. The pressure-variations between the inside and the outside of the outlet-bend have not been shown as they do not affect the main problem.

The next example—type B—will be taken as having a 15-foot effective head, and with outlet-area equal to the passage-area, that is, there is no contraction at the outlet. The vertical limb is short and is covered by the straight lengths taken with the bends. The results of the calculations are given in Tables VI and VII, and the losses are plotted on the developed diagram (Fig. 10, Plate 1).

TABLE VI.—SIPHON (TYPE B) WITH 15-FOOT HEAD.

Losses in terms of diameters and $\frac{V^2}{2g}$.

$\left(\text{Loss per diameter} = 0{\cdot}0168\ \frac{V^2}{2g}.\right)$

Section.	Velocity.	Length as number of diameters.	Extra length for bend.	Total length for loss.	Length adjusted for velocity.	Loss of head.
Entrance passage	$\frac{V}{2}$	2	—	2	1·0	$0{\cdot}02\ \frac{V^2}{2g}$
Upper bend	V	6	26·0	32·0	32·0	0·54 ,,
Lower bend	V	5	7·0	12·0	12·0	0·20 ,,
Discharge .	V	—	—	—	—	1·125 ,,
	Totals	13	34	—	45	$1{\cdot}885\ \frac{V^2}{2g}$

For a 15-foot head, $\frac{V^2}{2g} = \frac{15}{1{\cdot}885} = 7{\cdot}9$ and $V = 8\sqrt{7{\cdot}9} = 22{\cdot}4$ feet per second.

The discharge head $= 1{\cdot}125 \times 7{\cdot}9 = 8{\cdot}9$ feet, leaving 6·1 feet hea[d] available for overcoming friction.

TABLE VII.—SIPHON (TYPE B) WITH 15-FOOT HEAD.

Losses in feet of head.

Section.	Friction-loss and discharge-loss.	Cumulative friction-loss.	Velocity-head.	Total cumulative loss of pressure-head.
Entrance passage . .	$6{\cdot}1 \times \frac{1}{45} = 0{\cdot}13$	0·13	0 to 7·9	8·03
Upper bend:				
1. To 90° . . .	$6{\cdot}1 \times \frac{10{\cdot}7}{45} = 1{\cdot}45$	1·58	$7{\cdot}9 \times 1{\cdot}3 = 10{\cdot}3$	11·88
2. To 180° . . .	,, ,, = 1·45	3·03	$7{\cdot}9 \times 1{\cdot}3 = 10{\cdot}3$	13·33
3. 180° + 2 diam. .	,, ,, = 1·45	4·48	$7{\cdot}9 \times 1{\cdot}0 = 7{\cdot}9$	12·38
Lower bend . . .	$6{\cdot}1 + \frac{12{\cdot}0}{45} = 1{\cdot}62$	6·10	$7{\cdot}9 \times 1{\cdot}125 = 8{\cdot}9$	15·00
Discharge	8·90	6·10	8·9	15·00
	Total 15·00			

At the outside of the upper bend, the velocity-head would be $7{\cdot}9 \times 0{\cdot}46$ $= 3{\cdot}6$ feet, a decrease of $10{\cdot}3 - 3{\cdot}6 = 6{\cdot}7$ feet from the head used for plotting the line of loss of head, and at the inside it would be $7{\cdot}9 \times 3{\cdot}63$ $= 28{\cdot}7$, an increase of $28{\cdot}7 - 10{\cdot}3 = 18{\cdot}4$ feet over the head used for plotting. The maximum loss of head at the crest, including friction-loss, would therefore be $11{\cdot}88 + 18{\cdot}4 = 30{\cdot}28$, which approaches the limit even more closely than the example of type A.

The spouting-velocity on a 15-foot head would be $8\sqrt{15} = 31$ feet per second. The coefficient of discharge on the passage- and outlet-velocity is therefore $\frac{22{\cdot}4}{31} = 0{\cdot}72$.

The discharging capacity would be $4 \times 4 \times 22{\cdot}4 = 358$ cusecs, which is about the same as for the example of type A. They would be identical for equal limiting conditions at the crown-bend.

Consideration may now be given to the principal factors which influence efficiency in design. It is clear that the greater the friction-losses, the smaller will be the head available for generating velocity. Examination of Tables VI and VII will show the great effect which sharp bends have in increasing friction-losses, the increase in equivalent passage-length being from 18 diameters to 53 diameters in the example of Type A and from 13 to 45 in example Type B. By using easier bends the losses can be very much reduced and a better coefficient of discharge obtained.

An adverse effect of a different character is produced by the use of a sharp bend at the crown. This is the great local drop in pressure-head at

the crest on the inside of the bend, which uses up a large part of the available atmospheric head. This is clearly shown in Figs. 9 and 10, Plate 1, for sharp bends of $R/D = 1{\cdot}06$. This adverse effect can also be largely eliminated by using an easy bend. Further improvement, where circumstances permit, can be obtained by widening the passage at the crown and reducing the depth, the effect of these modifications being to increase the ratio R/D which determines the magnitude of the bend-loss and the additional velocity-head, both of which are reduced by increase of R/D. If in the crown-bend R/D had been 2·5 instead of 1·06 the friction-loss would have been reduced and the head available for generating velocity in both types of siphon would have been increased, with corresponding increase in the discharging capacity. The total head could also have been increased over the 40 feet and 15 feet assumed, with further increase in the discharging capacity.

It will be realized that in general the crown-bend is the part which imposes the strictest limitations on the capacity of a siphon. Conditions will normally be satisfactory if the mean velocities for quantity at crown-bends as arrived at by the methods of calculation detailed herein do not exceed the values given in the following Table VIII.

Table VIII.—Limiting Values of Mean Velocity (feet per second) at Crown-Bends of Siphons for various Altitudes and Bend-Ratios R/D.

Height above sea-level: feet.	Crown-bends of ratios R/D.				
	0·8	1·0	1·5	2·0	2·5
0	15	20	27	30	32
2,000	14	19	25	29	31
4,000	13	18	24	27	29
6,000	13	17	23	26	28

The values in the above Table allow for a margin of about 6 feet on the available atmospheric head. It will be seen that the possible safe capacity with an easy bend having $R/D = 2{\cdot}5$ is more than twice as great as the capacity with a bend of very sharp radius having $R/D = 0{\cdot}8$.

A model-test carried out on a reduced scale in ordinary atmospheric pressure will fail to give an indication by its operation of the troubles which may arise in the full-size siphon from too high a vacuum. The reason is that in the model only a fraction of the atmospheric head would be involved and the necessary scalar relationship in respect of operating pressure is not maintained. It would be possible in a laboratory, no doubt with some difficulty and at considerable expense, to have a vacuum-tank with observation-windows and an access door, within which the arrangements for a model-test could be set up, the test being carried out under observation from the outside with the atmospheric pressure reduced in proportion to the scale of the model. Suitable means would be required for passing in

the water-supply, exhausting the tail-water and maintaining the reduced pressure in the tank. If such a test were made on a model in which scalar relationship for roughness was also maintained, then the operating conditions in the model would be closely comparable with the operating conditions in the full size; and if too high a vacuum occurred at the crown-bend so as to modify the discharge, the effect would be apparent in the results from the model.

Summary of Approximate Laws of Flow Applicable to Large Pipes, Conduits, Tunnels, Bends, and Siphons.

(1) In straight pipes and conduits suitable approximate formulas for flow (foot-second units) are,

$$V = \frac{1{\cdot}486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}},$$

and

$$S = \frac{V^2 n^2}{2{\cdot}2 R^{1{\cdot}333}}.$$

In metre-second units the formulas are

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}},$$

and

$$S = \frac{V^2 n^2}{R^{1{\cdot}333}}.$$

(2) Values of the coefficient of roughness, n, for use in the above formulas are given in Table IX in the Appendix for surfaces of various characters.

(3) The coefficient, n, in the formulas given under (1) is directly related to the "absolute roughness" of the surface, the "absolute roughness", k, being approximately proportional to the sixth power of n.

(4) Comparative values for the "absolute roughness" are given in Table X in the Appendix.

(5) In straight pipes the flow is not uniform, being less at the sides and more at the centre than the mean velocity V_q. The variation from uniformity increases with increase in the relative roughness.

(6) In pipes of different diameters, flowing with roughness effect fully developed, there will be approximate similarity of flow under comparable conditions when scalar relationship is maintained between "absolute roughness" and diameter; that is when

$$d_1/n_1{}^6 = d_2/n_2{}^6 \text{ or } d_1/k_1 = d_2/k_2.$$

(7) The mean kinetic energy in straight pipes is more than $\frac{V_q{}^2}{2g}$ (V_q = mean velocity for quantity), and when the flow is steady can be

evaluated by arithmetical integration from the velocity-curve taken across a diameter. In pipes of the same diameter the excess kinetic energy over $\frac{V_q^2}{2g}$ increases with increase in the coefficient of roughness.

(8) If increase in the kinetic energy without increase in the quantity flowing is induced by other means than the roughness of the pipe, this increase is accompanied by further increase in the frictional loss. Such other means producing increase of kinetic energy are (*a*) vortex-motion imposed by flow around bends and (*b*) jet-conditions (vena contracta) produced at the outlet-ends of sharp bends.

(9) Losses in bends follow similar laws to the losses in straight pipes, but are on a higher scale. They vary directly as V^2 and n^2 and inversely as $R^{1 \cdot 333}$.

(10) Losses in bends as ascertained by experiment or test should be expressed as the equivalent of the loss in a certain number of diameters of straight pipe having the same cross section and roughness of surface and the same velocity. Under variations of scale, velocity, and roughness, the number of diameters of actual straight pipe which represents the loss will remain constant.

(11) The formula $S = \frac{V^2 n^2}{2 \cdot 2 R^{1 \cdot 333}}$ (foot-second units) will be applicable for variations of V, n, and R in the case of losses in bends. When bend-losses expressed in numbers of diameters as indicated in (10) are converted into the corresponding length L in feet of actual straight pipe, the total loss will be LS.

(12) Table II and *Fig. 8* (p. 468) give losses for various bends on 4-inch square pipes expressed as numbers of diameters of straight pipe. These losses will be applicable to bends on pipes or passages of square section. It is considered that the same losses may be used for circular pipes for the purpose of arriving at conservative discharging capacity.

(13) The laws for bends in pipes apply also to bends in siphons.

(14) The flow in a siphon is equivalent to the flow through a straight pipe of the same cross section but of greater length. The increased length for purposes of calculation can be obtained by adding the appropriate extra length for each bend and making suitable adjustments where changes of section produce changes of velocity. Where there is a constant quantity flowing but variation of cross section produces changes of velocity, the loss varies as $V^{2 \cdot 67}$.

(15) Siphonic flow is dependent on atmospheric pressure, which imposes limitations. Methods of keeping within the limitations, in particular the avoidance of vacuum due to the high vortex-velocity at the crest, are indicated in the Paper.

(16) The results of model-tests on siphons can be correlated approximately to the results to be expected on full size, provided that proper

allowance is made for change of scale and change of roughness by means o the pipe-flow formula, and that account is taken of the limitations arisin from atmospheric pressure. The size of the passages in the model and th nature of the surface should be such that the losses vary as V^2 and not as a power of V less or more than 2. This condition requires that the minimum diameter of passage should not be much less than 2 inches. To obtain th correlation, it is necessary that the coefficient of roughness, N_m, for th surfaces of the passage in the model should be known or ascertained b experiment, and that the coefficient N_a for the full-size apparatus shoul also be known.

(17) A model tested under atmospheric pressure will give no indicatio as to whether or not high-vacuum conditions with adverse effect on the flow may occur in the full-size siphon. In order to give such indications th model would require to be tested under suitably lowered air-pressure.

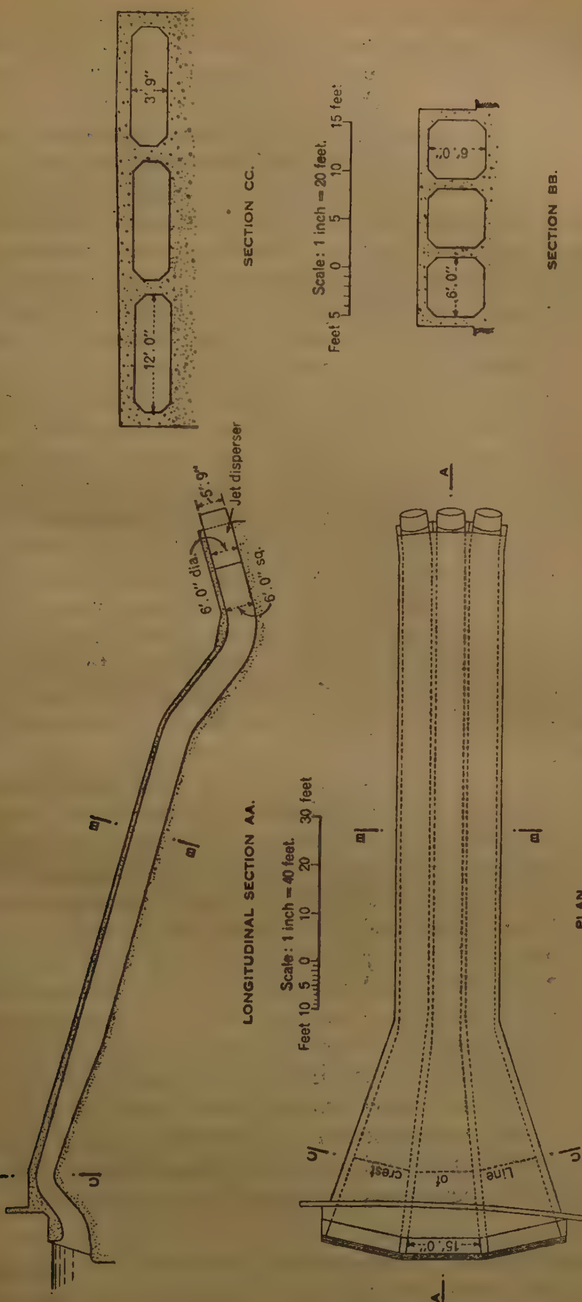
Siphon Installations and Designs.

A short description of two siphon installations designed by the Autho may be of interest.

Figs. 11 shows the arrangements of a battery of three siphons installe at the Loch Doon dam in the Galloway Water Power Scheme, Scotland. The outlet-legs are arranged to follow the slope of the ground and ar longer than usual. The normal passage is 6 feet by 6 feet with splaye corners, and the three are constructed together in reinforced concrete The arrangements at the inlet-end and over the crest are somewha unusual. The inlet-mouths are wide and deep to give large area and smal entrance-velocity. Gradual contraction of area proceeds from the inle over the crest until the normal 6 feet by 6 feet passage is reached on the downstream leg. This contraction ensures satisfactory hydraulic flow under the varying form of the cross section. The depth of the section is reduced to a minimum of 3 feet 9 inches over the crest, the width of the passage at this place being about 12 feet. The extra width at the crest provides that double the water will pass over with a given depth in priming as compared with that which would pass if the normal width of 6 feet had been maintained. The speed of priming will therefore be increased The reduction of the depth of passage increases the bend-ratio R/D, and reduces the kinetic energy and the maximum drop in the pressure-head.

The arrangements at the outlet-end are also rather unusual. The outlet end of the passage is formed of circular pipe converging to 5 feet 9 inches in diameter at the outlet. This part is constructed in metal and is tilted upwards and extended so as to provide a sealed outlet during priming A metal diaphragm with disperser-vanes of screw-form is included at the converging part of the outlet. The vanes have varying pitch, flat at the outside and steep at the inside. Their effect in conjunction with the convergence is to impart free vortex-motion to the issuing water so that o

Figs. 11.



Siphons at Loch Doon Dam.

emerging from the pipe the more rapidly spinning central layers bur outwards and break up the water into drops which are rapidly retarded l passage through the air. The upward slope of the outlet increases th length of the air-passage to the river which lies at a lower level. Th estimated capacity of each unit is over 900 cusecs with an issuing velociti of about 36 feet per second, under a total head from reservoir to outlet about 33 feet. The crests of the siphons are stepped in elevation so tha they come into action successively under a continuing rise of the reservoir

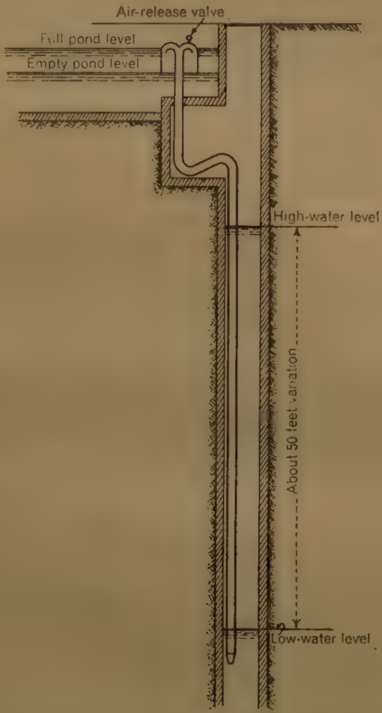
Another application of a siphon, but for a special purpose and und unusual conditions, occurs in the Galloway Scheme at a construction sha on the Glenlee tunnel, where it was desired to collect additional water b taking in a small stream which passes the shaft but does not flow to th reservoir. The water might have been poured down the shaft but this ha been found in other cases to be objectionable in that much air in very fi bubbles gets into the water and finds its way into the tunnel. Any suc air passing on into the pipe-lines gets compressed to high pressure an expands suddenly in the release of pressure in passing through the turbin and if the air is in considerable quantity there is detrimental effect on th efficiency of operation. It was felt that if a pond could be arranged on th stream and emptied periodically by a siphon with the outlet leg carrie down the shaft and the bottom of the pipe always submerged, the wate would be passed in without air except for a little during priming an breaking of the siphon. The problem, however, was to design a sipho which would work and flow full under great variations of head arising fro a fluctuation of reservoir-level of 43 feet and further fluctuation of the lev in the shaft owing to varying hydraulic gradient in the tunnel. The tot range of level at the shaft was thus over 50 feet. The problem was succes fully met by providing a nozzle with suitably proportioned contraction a the bottom end of the pipe in the shaft. The arrangement of the sipho is shown schematically in *Fig. 12.* The bellmouth-entrance of the pipe i in a chamber forming an extension of the pond. The entrance is covere by a hood and a cylindrical casing, the bottom end of which is about 3 fe below the bellmouth. The siphon-pipe passes down vertically from th bellmouth and is sealed through the floor of the chamber below which it i supported in a recess and offset by means of an S bend to enter the shaf in which it is supported in a vertical position. The S bend facilitate priming. The proportions of the inlet-mouth, diameter of pipe, and dia meter of outlet-nozzle are so arranged that under all conditions th pressure at no point in the system reaches absolute vacuum. A small ai valve is provided on the hood over the bellmouth-inlet. The course o operations after the pond has been emptied and the siphon unsealed is a follows:

(1) Gradual filling of the pond by inflow from the stream. Wate rises inside the cylindrical casing and as it rises the air dis placed is pushed out through the air-valve.

(2) Priming commences when water begins to flow over the rim of the bellmouth. Slight reduction of pressure closes the air-valve and priming is rapidly completed. During priming some air is carried into the water in the shaft.

(3) On completion of priming the pipe flows full and gradually empties the pond, the water being discharged in a solid stream below the standing-water level in the shaft.

Fig. 12.



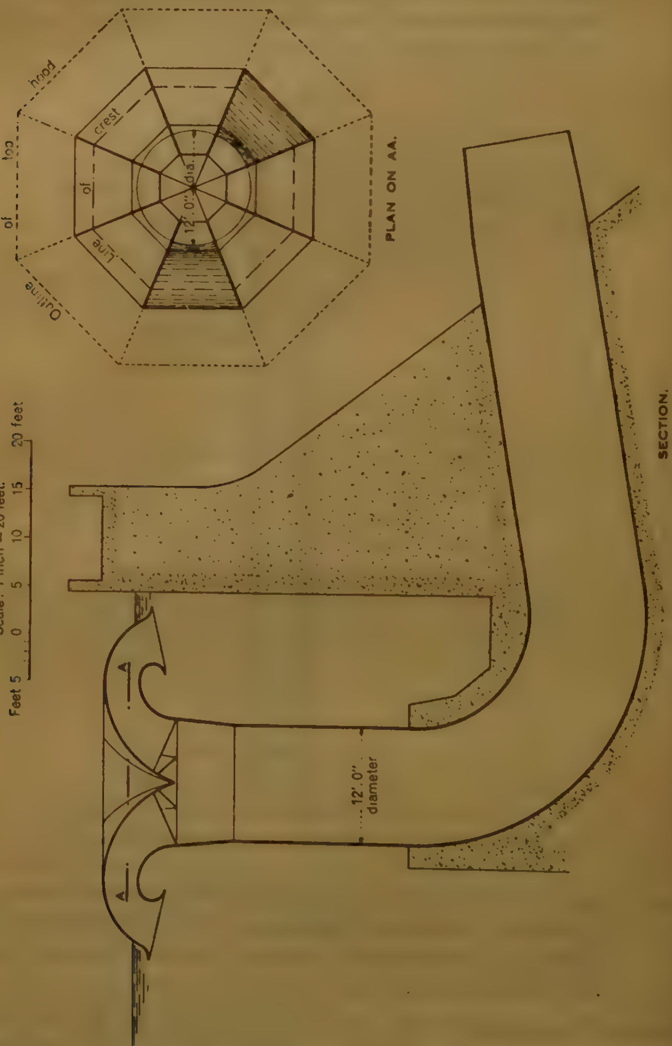
SIPHON INSTALLED IN CONSTRUCTION SHAFT OF TUNNEL.

(4) When the water in the head-chamber falls to the bottom of the cylindrical casing, air is drawn in with the water for a brief period, the seal is rapidly broken, flow stops and the filling of the pond recommences.

This small installation, with an average capacity of about 12 cusecs, dmirably serves the purpose for which it was designed. The capacity aturally varies with the head and is smallest when the reservoir is high, eing then about 60 per cent. of the maximum. There is, of course, little eed for collecting additional water when the reservoir is full. During ertain periods when the flow in the stream is high, the siphon works

continuously. The power-station operates on peak load and does n generally run during the night. When the station is not running the wat taken in by the siphon flows back through the tunnel to be stored in t reservoir.

Figs. 13.



SIPHON FOR LARGE DISCHARGE.

A siphon arrangement devised by the Author and intended particularl for large discharges is indicated in *Figs. 13*. The elements are a vertic shaft within the reservoir connected by a bend to a horizontal or incline passage with outlet at the downstream side of the dam. The outlet ma

e plain or contracted as required to suit the head, and may where necessary e furnished with a disperser. On top of the shaft there is a bellmouth-ntrance with a shaped hood. Proportions are arranged so that the inlet-elocity is small, the velocity over the crest is moderate and much less han the velocity in the shaft and passage, and there is gradual and con-inuous increase of velocity from the inlet-mouth to the top of the circular haft. The length of crest is multiplied several times as compared with iphon forms usually adopted hitherto so that priming can take place with relatively small depth over the crest. By suitably tilting the outlet assage complete sealing of the outlet during priming can be arranged. he siphon of *Figs. 13* has been drawn to suit a diameter of shaft and passage f 12 feet and a head of 36 feet. The calculated capacity is about 4,000 usecs.

Problem of the Simple Surge-Tank.

The simple surge-tank is a tank of constant horizontal cross section aving a large communicating passage to the pressure-aqueduct so that here is no throttling producing modification of pressure as the water nters or leaves the tank. Application of the pipe-flow formula enables a etter realization of the conditions affecting design to be obtained and ffects simplification of the calculations. Where pressure-aqueducts are ong a surge-tank is required and should be located as near to the power-tation as is practicable. Its first object is to limit the fluctuations of ressure and head under varying load-conditions and to enable the turbine-overnor to take control and adjust the flow to steady conditions suitable o the demand for power. If the tank is too small in cross-sectional area luctuations will be large and the action of the governor will tend to increase he fluctuations and maintain "hunting". If the tank is large enough to e on the point of incipient stability, control will be difficult and fluctua-ions, if they are damped out, will only die away slowly. If a sufficient nargin is provided over the requirements for initial stability, the governor vill take control and reduce fluctuations rapidly to steady flow. The xtent to which swinging conditions may persist in the absence of governor ontrol is not generally realized. At the Glenlee power-station of the Galloway Water Power Company, the pressure-tunnel is about $3\frac{3}{4}$ miles ong and the surge tank 24 feet in diameter. If the station is rapidly shut lown the water rises in the surge-tank above the reservoir-level, then falls elow the reservoir-level and continues swinging with reversal of flow in the unnel for 2 or 3 hours. A surge-fluctuation produced by change of oad when the station is running is on the other hand quickly reduced to teady flow by the action of the governor.

Thoma's formula is generally used to ascertain the dimensions which

will produce incipient stability, and is in the form

$$F_m = \frac{A \, . \, L}{2g \, . \, C \, . \, H} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5$$

where F_m denotes the horizontal sectional area of the tank;
A denotes the cross sectional area of the aqueduct;
L denotes the length of the aqueduct from the reservoir to surge-tank;
C is a friction coefficient $= \frac{f}{v^2}$;
v denotes the velocity in the aqueduct;
f denotes the total friction-loss in the aqueduct for velocity
and H denotes the minimum working head on the turbines.

By application of the pipe-flow formula, the coefficient C is found to $\frac{n^2 L}{2{\cdot}2R^{1{\cdot}33}}$ and Thoma's formula therefore reduces to

$$\frac{T^2}{D^2} = \frac{0{\cdot}0054 D^{1{\cdot}33}}{n^2 H},$$

and

$$T = \frac{0{\cdot}074 \; D^{1{\cdot}67}}{n \, . \, H^{0{\cdot}5}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

where T denotes the diameter of the tank and D denotes the diame of the aqueduct.

It is seen that the minimum diameter of tank for stability varies direct as the 1·67th power of the diameter of the aqueduct, and inversely as t roughness-coefficient n and the square root of the head, but does not depe on the length. The salient features of the result are the very rapid rate which size of tank requires to increase with increase of diameter of t aqueduct and the fact that the smoother the aqueduct is made the larg must be the diameter of the tank. Thus, while increase of smoothn decreases friction-loss and adds to the output of power, it also adds to t cost of the surge-tank. The head H to be used in the formula is t lowest operating head on the station and the value of n should be taken point or so below the value expected to be right for the friction-lo calculations.

The value of T as calculated by formula (6) is the minimum diameter f stability and a margin is required to ensure satisfactory operation. three surge-tanks in the Galloway Water Power Scheme the margin pr vided was about 50 per cent. on the area or about 25 per cent. on t calculated minimum diameter and this provision has been found sat factory.

The Paper is accompanied by fifteen sheets of diagrams, from some which Plate 1 and the Figures in the text have been prepared, and by t following Appendix.

APPENDIX†.

TABLE IX.—VALUES OF COEFFICIENTS OF ROUGHNESS, n, FOR USE IN FORMULA

$$V = \frac{1{\cdot}486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \text{ (FOOT-SECOND UNITS).}$$

Character of Surface.	*Value of "n".*
ock surface, very rough	0·04–0·06
ock tunnel, surface trimmed	0·025–0·035
ock tunnel, trimmed and invert lined with concrete	0·020–0·030
hingle, 6 inches, average size	0·03–0·04
ravel, 3 inches, average size	0·027
ine gravel anals in fine earth	0·0225
ine gravel with much sand oughly-squared masonry	0·020
ell-squared masonry ough concrete	0·017
ld encrusted pipes	0·014–0·0165
apped riveted pipes, smooth coating	0·013–0·0165
rdinary concrete, from rough forms	0·013–0·0145
ressed masonry mooth flush-pointed brickwork ery well finished concrete	0·0125–0·013
iveted pipes, plates flush inside, rivet-heads flattened, smooth coating	0·012–0·013
mall concrete and earthenware pipes, clean	0·012–0·013
oncrete-lined tunnel, and reinforced-concrete conduits, from steel forms, new	0·0115–0·0125
ew cast-iron pipes, smooth coating elded steel pipes, smooth coating	0·0110–0·0125
moothed plaster of sand and cement	0·0115–0·012
arge machine-made concrete pipes, clean	0·010–0·0115
ew planed wood (wood-stave pipe)	0·010–0·011
moothest fine cement plaster	0·010
mooth asbestos-cement pipes	0·0086–0·010
eak, planed and sandpapered (in models)	0·0084
rawn brass and copper tube	0·0067
ery smooth varnish on smoothed wood	0·0064

† Abridged. (Note on "Requirements for Ascertaining Loss of Head in Large ipes" omitted.) The MS. may be seen in the Institution Library.—SEC. INST. C.E.

TABLE X.—COMPARATIVE VALUES OF COEFFICIENT, n, AND "ABSOLUTE ROUGHNESS."

"Absolute roughness" may be visualized as the average diameter of projecti rounded grains forming the roughness of a uniform surface. It is assumed that t "absolute roughness" is the only factor producing resistance to flow and that t effects of irregularities of shape, form, and cross section of conduit (which wou produce additional resistance) are excluded. "Absolute roughness" varies as t sixth power of n.

Roughness-coefficient n:	0·025	0·022	0·020	0·015	0·014	0·013	0·0125	0·0120	0·0115	0·01
Absolute roughness: inches.	3·3	1·6	0·88	0·16	0·105	0·066	0·052	0·041	0·032	0·02
Absolute roughness: centimetres.	8·5	4·0	2·2	0·40	0·26	0·16	0·13	0·105	0·082	0·06

Roughness-coefficient n:	0·0105	0·0100	0·0095	0·0090	0·0085	0·0080	0·0075	0·0070	0·00
Absolute roughness: inch.	0·019	0·014	0·010	0·0072	0·0051	0·0036	0·0025	0·0016	0·00
Absolute roughness: centimetre.	0·047	0·035	0·025	0·019	0·013	0·009	0·006	0·004	0·00

Note.—The above Table is approximately correct at the two highest values whic correspond to the resistances of regular channel beds consisting of projecting rounde pebbles. The Table is also closely correct in the region from n = 0·0085 to n = 0·01 which covers the range of artificial sand-grain surfaces in Nikuradse's experiments.

Discussion.

The Author explained that the Paper was largely based on information furnished in certain previous Papers presented to The Institution, to which reference was made on p. 451. Those Papers related especially to the question of flow through siphons, and since their presentation he had gone further into the matter with particular reference to the question of flow in large pipes and large bends, and the relation which those considerations would have to the question of siphons. In addition to the Authors of those previous Papers, he acknowledged the work of certain friends who had furnished some of the information which was given in Figs. 2, Plate 1. His thanks were due to the Galloway Water Power Company and to Sir Alexander Gibb and Partners for some of the information taken from the Galloway Water Power Scheme, and also to the British Headometer Company, Mr. W. T. Halcrow, and Messrs. Balfour Beatty & Company for information regarding the velocity-diagrams on flow in large pipes.

The President, in moving a vote of thanks to the Author, said that the Paper was of great interest. With regard to the coefficient of roughness, the Research Committee of The Institution had set up a Velocity Formulas Sub-Committee which had not yet reported, but Mr. A. Bailey was carrying out experiments at Teddington on its behalf on 6-inch diameter concrete, steel, cast-iron, and asbestos pipes. It was proposed that those experiments should be extended to bends, because, as was pointed out in the present Paper, the information regarding bends was scanty.

Dr. Herbert Chatley said that, as a member of a Panel of the Committee to which the President had referred, he found great difficulty in saying anything very definite about the present Paper, because it covered the work of that Committee and was a very praiseworthy and ambitious attempt to forestall the ultimate result of that Committee's researches. It was a very excellent idea of the Author's to endeavour to discriminate clearly between the resistance in curves and in straights, because until those two were clearly separated it was really impossible to make any proper calculations of pipe-resistance. The work done in laboratories generally involved long straight pipes, and as few such pipes occurred in practice the practising engineer found it very difficult to apply theoretical results.

The Author referred first to the Chezy formula, and suggested that the deviations of the coefficient of discharge in that formula were simply due to scale-effects. Dr. Chatley did not think that that was exactly

true; even with pipes of what might be termed "limiting smoothness"- a smoothness so great that even a large difference in the pipe-diamet made a very small difference in the roughness-ratio—there was still very large change in the coefficient of discharge, which was due principal to viscosity-effects. The Author did not consider the effect of viscosit which expressed itself through temperature-effects. In many of th standard data on the matter temperature was not mentioned, and it ther fore became extremely difficult to ascertain what the viscosity-effec really were.

The Author then considered the Strickler formula, which, as th Author said, was really the Manning formula, and was probably bett known under that name. There were several other formulas which we fairly generally accepted and closely resembled it. There was the Forch heimer formula, which used the power of $\frac{7}{10}$ instead of $\frac{2}{3}$, and was no very popular in Germany, and there was Mr. Gerald Lacey's formul which used a power of $\frac{3}{4}$. All three of those formulas, which were com paratively simple, violated a certain rule or relation between the exponen which had been discussed at great length ; in Dr. C. F. Colebrook's Paper[1] it was stated, and apparently demonstrated with considerable thorough ness, that that relation between the exponents, in which the exponent the radius added to the exponent of the velocity, when expressed in term of slope, totalled 3, was true for smooth pipes but was not true for roug pipes. In that Paper there was given a higher value for rough pipes. that were the case, then none of the three formulas referred to was pe fectly correct, but the Author might be quite right in saying that his was good working formula, and certainly it had the advantage of simplicity The remarkable researches made by Professors Prandtl and von Kármán especially in developing Nikuradse's results, could not, however, be pu entirely on one side, and there would undoubtedly be some uncertaint regarding the value of n, although it might not differ very much from constant value. When it was remembered that in all the work in questio it was extremely difficult to measure discharges or velocities within 5 o 10 per cent., it appeared to be extremely probable that the coefficients o roughness and discharge were also uncertain by similar percentages.

On p. 461 reference was made to the development of velocity aroun bends, and to its distribution. As far as Dr. Chatley knew, no one ha ever yet actually measured that distribution with any particular accuracy The Author suggested that the general tendency was to form a free vortex and on the basis of that assumption he showed the manner in which th kinetic energy changed with the sharpness of the bend and the manne in which the velocity varied across the diameter of the pipe. Since i was a free vortex, it necessarily diminished towards the outside of th

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[1] "Turbulent Flow in Pipes, with particular reference to the Transition Regio between the Smooth- and Rough-Pipe Laws." Journal Inst. C.E., vol. 11 (1938–39 p. 133. (February 1939).

bend. From that the Author deduced that the path of maximum kinetic energy followed the curve shown in *Fig. 7* (p. 462). Dr. Chatley suggested, however, that that might not be exactly so. If the whole of the discharge around the curve followed a free vortex there would be no loss of energy, because it was the essential feature of a free vortex that the particles were, so to speak, turning backwards as their whole mass turned in the opposite direction, so that there was no spin and the free vortex could recover itself without any internal force or "molecular spin." In so far as there was friction the vortex ceased to be free. Whilst he was perfectly prepared to admit that some part of the flow was a free vortex, he was quite convinced that, at any rate in open channels, and he thought almost certainly in pipes, a certain part of the flow was not a free vortex, so that he thought that there was a tendency at a 180-degree bend for the flow to

e bend started,
ould then be a
ortex, in which

the water was caused to spin, on the inside of the bend; there was a cushion where the water might even move backwards, or at any rate have its velocity greatly reduced, and there was a very great reduction of pressure. Bernoulli's theorem only applied to pure streamline flow, and was modified in turbulent streams and in eddies. In so far as the flow around the bend might be a forced or partially forced vortex, and not a free vortex, there would be a loss of energy, so that the criterion of free or forced vortexes, or the distribution between those two forms, was also an indication of the loss of energy at the curve.

Concurrently, the Author referred very forcibly to the question of the difference between the actual kinetic energy and that measured by the square of the mean velocity. That was what electrical engineers called the "root-mean-square effect," and undoubtedly it was a rather neglected aspect of the research.

The study of bends, in Dr. Chatley's view, had been rather neglected. He had suggested elsewhere that there was a tendency for the loss of energy in the bends to be comparable with the frictional loss in a long length of irregularly-curved pipe or channel, and that fact was indicated also by the Author's figures. One point which arose in that connexion was the use of non-circular bends. In connexion with the design of drag-suction dredgers, he had found that it was worth while, even though it was inconvenient for the constructor and designer, to put in non-circular bends in places where resistance should be reduced. The reduction in resistance was almost certainly due to the fact that in a non-circular bend a larger proportion of the flow round the bend was of the free-vortex type than in the circular bend.

Mr. B. M. Hellstrom remarked that he would confine his remarks to the formula for straight pipes quoted on p. 453, namely, $V = KR^{\frac{2}{3}}S^{\frac{1}{2}}$. From the point of view of the practical engineer that formula was very

simple and easily handled. His firm had used Manning's formula i 15 years, and had so far found it satisfactory. The formula could written $V = KR^{\alpha}S^{\beta}$, and if K depended only on the roughness of the pi the indices α and β could be regarded as constants. The Author said th for a wide range of conditions it had been found that $V \propto R^{0 \cdot 67}$. Certa experiments with which Mr. Hellstrom had been associated agreed ve well with that result. With regard to the value of β, however, it w quite obvious, as Dr. Chatley had also pointed out, that the velocity its had an effect on the value of K if β were taken as $\frac{1}{2}$. For one and the san pipe $V \propto S^{\beta}$. For wooden pipes Mr. Hellstrom had found that $\beta = 0{\cdot}5$ for concrete pipes $\beta = 0{\cdot}52$, and for unlined rock tunnels $\beta = 0{\cdot}45$. I Manning's formula for smooth pipes the value of K would rise with increase velocity, whilst for an unlined tunnel the value of K fell with increase velocity. If K depended upon the roughness only it would in one and th same pipe be constant for any particular velocity. He had tried to ove come that difficulty by adjusting the value of K in regard to the velocity.

It was very interesting to hear what the President had said about th experiments which were being carried out for the Research Committe and he wondered whether there would be any opportunity of also measurin the velocity in unlined rock tunnels, in order to obtain coefficients coverin a wide field.

Mr. E. R. Howland said he had found a number of points of grea interest in the Paper. With regard to the distribution of velocity i pipes, he thought that the reason that the curve shown in Fig. 2 (c), Plate 1, showed so little effect of roughness was that at the time that th test was made the pipe was perfectly new.

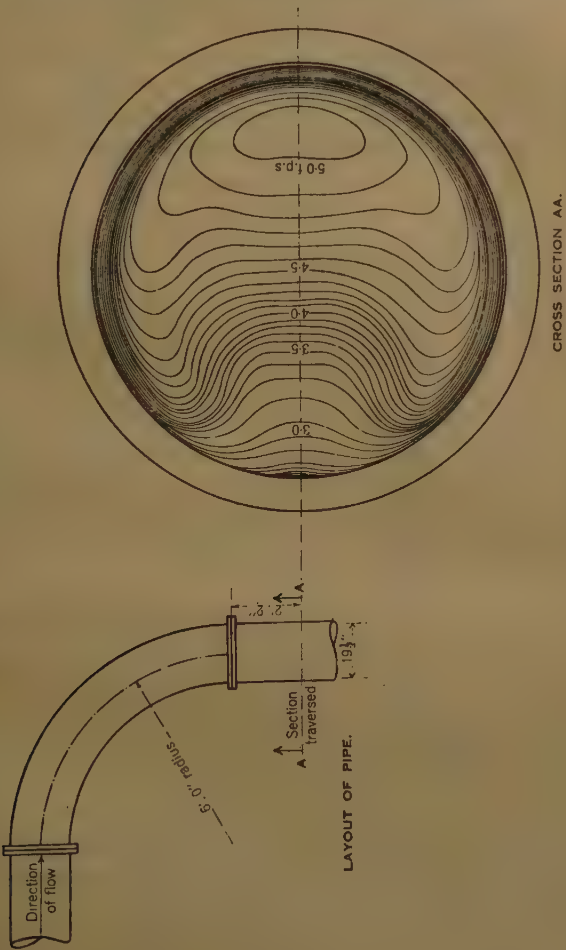
In Table I (p. 460) the third column was headed "Excess of side pressur over mean pressure." The figures appeared to have been arrived at o the assumption that pressures at points across a pipe varied by an amoun equal to the velocity-head. Consideration of the well-known type o Pitot tube taking the static pressure at the side of the pipe would sho that that was not so. If it were, the differential pressure produced woul be the head due to the velocity at the side of the pipe, and that woul remain constant no matter where the impact-orifice was placed, for althoug the impact-orifice would receive the full velocity-head, the total pressur on the orifice would have to be reduced by a similar amount to compensat for the loss of velocity-head at the point at which the orifice happened to be Actually a Pitot tube of that type showed the true distribution of velocity

The Venturi tube also assisted in showing that the assumption referre to was incorrect, for, if it were true, the pressure at the main, where th distribution of velocity was similar to that in an ordinary pipe, would b increased by the amounts shown in column (3), whilst the pressure at th throat, where the velocity was uniform except for a slight falling off a the wall, would receive only a very small increase. The effect would b that the Venturi-tube coefficients would be quite different from what the

were, or the extra Venturi head would have to be absorbed between the main and throat, and that would be very difficult to account for.

Figs. 14 showed the distribution of velocity in a 19½-inch pipe following a 6-foot radius bend, and showed the effect on the inside wall which was described by the Author. The Pitot-tube gaugings from which the

Figs. 14.



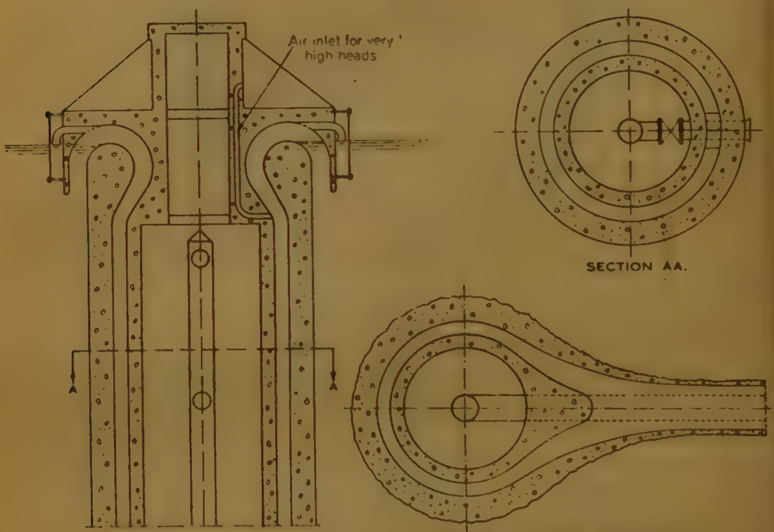
FLOW OF WATER UNDER PRESSURE: EFFECT OF 90-DEGREE BEND IN DISCHARGE-PIPE ON THE DISTRIBUTION OF VELOCITY.

drawing was made were taken in the plane of the bend at right angles to it and at 45 degrees to the bend, so that there was ample evidence from which to put in the contour-lines.

Mr. G. M. Binnie referred to the design for a siphon in a bellmouth form shown in *Figs. 13* (p. 486), and said that in that design the shallow lip of the inlet-cowl provided a means for air to enter and break the vacuum formed in the siphon when it had drawn the water in the reservoir down

to just below the level of the lip. If, on the other hand, the lip of th cowl were immersed several feet instead of a few inches, the formation c air-vortexes would be hindered more forcibly, and the intermittent makin and breaking of the siphon due to the lip becoming uncovered by wave action would also be prevented. If the lip were deeply immersed, som other method of preventing the siphon continuing to discharge after i had drawn the water down to top water-level would have to be provided That often took the form of air-inlet pipes, the inlet-ends of which wer bent down vertically and set at or near top water-level. At the sam time, in order that the air-inlet pipes themselves should not be subject t wave-action, they should be provided with a still-water well formed by

Figs. 15.



baffle-wall several feet deep. However, the adoption of a deeply sub merged lip and air-inlet pipes with still-water wells increased the lengt of the overhanging section of the siphon considerably and would be some what impracticable in the case of the Author's design.

In an article [1] by Mr. James Semple, Assoc. M. Inst. C.E., on Dunwa reservoir, there was a description of a valve-shaft constructed inside bellmouth-overflow shaft. That idea could with advantage be applied t siphons also, and *Figs. 15* showed how that combination might be con structed. The only obstructions in the space formed between the tw shafts were the draw-off pipes, and the concrete surround for those pipe could be streamlined so as to minimize the obstruction caused by them

[1] "The Dunwan Reservoir." *Civil Engineering* (London), vol. xxxiv (1939), p.

The junction of the two shafts with the tunnel was shown in *Figs. 15*; the discharge pipe from the valve shaft could be laid underneath the invert of the tunnel.

The cowl could be designed as a reinforced-concrete cantilever around the valve-shaft, which acted as a centre column. In addition, reinforced-concrete ribs could also be added, if necessary, to give additional support. The cowl could thus be supported in umbrella formation over the crest without any division walls or other obstructions to flow, and the whole unit would act as one siphon. The extent of the overhang of the cowl beyond the external diameter of the outer shaft was not very considerable. During construction, it could be supported on scaffolding cantilevered out from the outer shaft, suitable temporary recesses for the purpose being left in the exterior of the shaft. The construction of the air-inlet pipes with their still-water well also offered no particular difficulty. The still-water well could be formed by means of non-corrodible metal plates attached to bolts projecting outwards from the cowl, as shown in *Figs. 15*.

With regard to hydraulic considerations, the siphon could be designed on lines similar to those described by the Author for his design. The arrangement was shown in the sectional elevation. For very high heads, it might be preferable to limit the siphonic action to the upper part of the structure only by allowing free access of the air to some point in the shaft below the crest, as was shown on the right-hand side of the sectional elevation. In that case the head acting on the siphon would be equivalent to the difference of level between the water in the reservoir and the air-inlet in the shaft. Good priming was ensured for the latter case by the overhanging bend.

Mr. E. E. Morgan proposed to call the formula $V_q=KR^{2/3}S^{1/2}$ the fundamental formula, and $V=\frac{1}{n}R^{2/3}S^{1/2}$ and $V=\frac{1 \cdot 486}{n}R^{2/3}S^{1/2}$ the derived formulas for metre-second and foot-second units respectively. Those formulas were not of recent origin; in fact, according to Dr. Strickler, Gauckler produced a formula of the first type in 1867, although it was never used to any great extent at that time. As a matter of fact, the formula was considered correct only for gradients of 0·0007—about 1 in 1,400—or steeper, and flatter gradients required a different formula. For that reason the formula did not become very popular. Again, although Manning in 1889 deduced the same formula as the result of an investigation comprising one hundred and sixty experiments, he was careful to point out that at least two or three others had also arrived at the same result quite independently. A full description was given in Manning's Paper[1]. Manning gave his formula as $V=CS^{1/2}R^{2/3}$, which he called formula "V" and which was the fundamental formula given in

[1] "On the Flow of Water in Open Channels and Pipes." Trans. Inst. C.E. of Ireland, vol. xx (1891), p. 161.

the present Paper. Manning was, Mr. Morgan believed, the first to discover the relationship between C and $\frac{1}{n}$ given in formulas (1) and (2) of the present Paper, for, in his reply to the discussion which followed his Paper at that time, Manning stated " . . . it is worthy of remark that the value of the reciprocal of C (the coefficient in this Formula V) corresponds closely with that of n, . . . both C and n being constant for the same channel." It was thus seen that there was justification for referring to formulas (1) or (2)—they were the same formula, but expressed in different units—as Manning's formula. The effect was, however, somewhat confusing for purposes of reference, for not only was the fundamental formula generally known as Manning's formula both in Great Britain and abroad, but so also was the derived formula. It might perhaps help if the latter formula were referred to as " Manning's derived formula, based on Kutter's n," or, more simply, as " Manning's derived formula."

In 1897, Crimp and Bruges produced a set of Tables for velocity and discharge of pipes based on the formula $V=124R^{\frac{2}{3}}S^{\frac{1}{2}}$, and if the value 0·012, which was used for concrete pipes, were substituted for n in formula (2), the Crimp and Bruges formula was obtained. In 1911 R. B. Buckley advocated Manning's derived formula for use in India. In 1918, Professor H. W. King advised the same formula for use in the United States for open channels, and in 1929 he recommended it to be used for pipes as well. In 1923, Dr. Strickler's Paper was published[1]. Dr. Strickler recommended the same formula, and later Lindquist, in a special report " On Velocity Formulas for Open Channels and Pipes " presented at the World Power Conference sectional meeting in 1933, recommended Manning's formula in the form $V=MR^{\frac{2}{3}}J^{\frac{1}{2}}$, and gave a Table of values for M, both on the metric and foot bases, and also stated that " at preliminary computations the coefficient M should be given a value equal to the coefficient of roughness n given by Ganguillet and Kutter." That was really the same as Manning's coefficient. In the same report, Lindquist considered the von Kármán–Prandtl formula, which was $V=4\sqrt{2g}(\log \frac{R}{\epsilon}+1{\cdot}7)\sqrt{RS}$, and substituted certain roughness values for ϵ. Lindquist expressed that formula in the form $V=KR^aS^b$, and, for various values of roughness which were given in the report already referred to, he deduced a set of figures for a and b in which b was $\frac{1}{2}$ and a was practically $\frac{2}{3}$. That gave him the fundamental formula, which he recommended for use for conduits of cast iron, steel, or concrete made up in short lengths, as well as for open channels and rivers. In other words, it was a fairly useful formula. Mr. Morgan did not claim perfection for it. He had investigated a number of experiments on channels using Manning's formula, and, for comparison, he had made similar tests using

[1] Footnote ([1]), p. 452.

the von Kármán–Prandtl formula. If the roughness-values given in Table X of the Appendix to the present Paper were plotted on logarithmic paper against the corresponding values of n, a straight line was obtained; the figures which he worked out for ϵ were practically on that line for values of n as large as 0·025, and nearly so for very smooth surfaces. A line drawn through those points was not quite on the original line, but the deviation was very slight: it might mean that the index of R was 0·65 and 0·66 in some cases, instead of being $\frac{2}{3}$. Very smooth surfaces, however, were not met with frequently in drainage-work, because they did not stay smooth for very long.

Mr. Morgan did not claim that Manning's formula would give absolute accuracy for all conditions of surface, for all shapes of cross sections, and for all slopes. Such a formula, if ever evolved, would be so complicated to apply that no practical engineer would ever have time to use it. He did, however, consider that Manning's formula, whether in the fundamental or in the derived form, was not only easy to use, but was also one that gave accurate results within the limits of practical requirements and over a wide range of conditions, sizes, and slopes; for that reason he thought that the Author was entirely justified in using Manning's formula and Manning's derived formula for the purposes of his calculations.

The Author, in reply, said he appreciated very much the interest which had been taken in the Paper, particularly as he recognized that it was somewhat abstruse. Dr. Chatley had noticed that no reference was made in the Paper to viscosity. That was deliberate, and no reference was made anywhere in the Paper to the Reynolds' number either. He had purposely confined his investigation to the region of large pipes where the V^2 law was involved, and had not dealt with any of the partial-viscosity conditions where loss of gradient varied as $V^{1 \cdot 75}$, or as some power of V other than 2. When the experience which he had had in large pipes under varying velocity-conditions, where he had always found that there was no appreciable variation from the V^2 law, was correlated with the results of Nikuradse on small pipes of 1-inch, 2-inch, and 4-inch diameter for the sections of the investigation where the V^2 law applied, then over that complete range from small pipes to large pipes there was very close correspondence. He would not attempt to give any results for conditions which came partly within the viscosity conditions, because he considered that the only complete investigation which traversed the range from full viscosity to full turbulent flow was that of Nikuradse; if in the region of change from one to the other an attempt were made to obtain any simple formula where part of the loss was dependent on the Reynolds' number and part of the loss was dependent on the ratio of roughness to diameter, and the Reynolds number was not known to begin with, a very difficult problem indeed arose from the practical point of view.

He did not entirely follow Dr. Chatley's theory that with a free vortex flow there was no resistance. Far from it; investigations which he him-

self had made, which went perhaps a little beyond what was given in t. Paper, brought out as clearly as possible that the reason for high resistan in a bend was the high velocity past the inside of the bend. Wherev there was high velocity past the roughness protuberances of a surfa turbulences would be generated, and the power-rate at which turbulenc were generated varied as the cube of the velocity. If cubes of the veloci were integrated over the pipe-surface in a sharp bend, velocities bei assumed to follow the law of free-vortex flow, it was possible to arrive ve closely at the actual loss which did take place at such a bend, and there w no question whatever that with approximation to free-vortex flow, pr ducing high velocity at the inside and low velocity at the outside, the would be very little generation of turbulence on the outer face but enormous generation on the inner face, and correspondingly a very hi resistance produced on the inside of the bend. To his mind there w no doubt that the high resistance in a sharp bend was produced on t inside surface.

Dr. Chatley's remarks on the use of non-circular bends were of interes A good deal of further investigation might be carried out on methods decreasing the loss in bends. Dr. Chatley also referred to a point whic was of extreme importance and which was referred to in the MS., althoug not in the Paper as printed. It was that when the progress of the vari tions which occurred in the distribution of flow in a large pipe was studie it was realized that the bulk of the records which had been made hither on losses in large pipes could be approximate only. Taking a theoretic power pipe-line, at the top there was probably something in the nature of bellmouth inlet. At any bellmouth the effect was to produce at the com mencement a nearly constant velocity across the width of the pipe. As th water travelled along the pipe, the friction drag or resistance at the sid gradually modified the velocity, slowing it down at the sides and increasin it in the middle. After a travel of several hundred feet a steady conditio of flow might be reached, with a fully-developed velocity-curve. If a co traction occurred conditions were again encountered which produced pract cally uniform velocity, followed by gradual modification of the velocit distribution until another contraction, or a valve, or some other disturbin factor, was encountered. In large pipes there could be no doubt that length of 100 diameters or more would be required to even out the co ditions. The reason for that arose from the more rapid rate at which t mass and inertia of the water increased with increase of diameter compared with the surface-area which produced the resistance. believed that the total energies in the various filaments in a pipe were t same at any point on a transverse diameter, and if there were a high veloci in the middle and a low velocity at the sides, there would necessarily be variation of pressure as between the middle and the sides. If, therefo the pressure were measured by gauge at the top end of a pipe near a be mouth, the pressure read should correspond to the pressure of the strea

nd if measured lower down after development of the roughness-effect, he pressure would be influenced by the lower side velocity and would be igher than the mean pressure of the stream; the pressure-gauge differences n such a developing length would therefore not give a true measure of he loss of head. In any such pipe-line, even although the stretches were hundreds of feet long, there was a probability that the pressure-losses as etermined from gauges reading the side-pressures would not be correctly etermined. Even if the mean pressures of the stream were correctly etermined, it would be necessary to take account of the gain of kinetic nergy during movement downwards, which, although producing loss of ressure, was not a loss of energy, so that, unless in the case of exceptionally ong and regular pipes, he was convinced that the records hitherto obtained ere at best only very roughly approximate. That was why he had called he formula which he had suggested only an approximate formula. Until nethods were adopted to secure far better records with regard to actual oss than had hitherto obtained, he did not think that it would be possible o say that any particular formula was very near the truth so far as the ecords of large pipes were concerned. In the case of laboratory-tests enerally, it was quite possible to discard the developing portion of the ipe and to carry out the tests on a section where uniform steady flow ad been confirmed by actual testing, as was done in Nikuradse's tests. Jsually in a rough pipe of small size something like 30 diameters would ully develop the effects of the roughness, and after that there would be a onstant flow-condition, but in a large pipe much more than 30 diameters vas required, and when a particular limit of size was reached stable axial ow would not result, but the ultimate effect would be spiral flow. That vas frequently found in the larger classes of smooth pipes.

He thought that Mr. Howland's suggestion that the actual pressure cross the whole width of a pipe was exactly the same was worthy of some urther investigation. To the Author's mind it conflicted with Bernoulli's aw, particularly as the variations of velocity and pressure across a section t a bend, where the variations were large and readily measurable, had een found to follow the Bernoulli law.

Mr. G. M. Binnie made some remarks on the siphon shown in *Figs. 13* p. 486) and some references to certain details which were obviously not hown on the diagram, and showed (*Figs. 15*, p. 496) another method of orming a siphon, which the Author considered would be more complicated nd costly than the type given in the Paper. Mr. Binnie referred to the uestion of deep submergence. If he referred to the first example of a siphon iven in the Paper (*Fig. 12*, p. 485) he would see that that had a hood with leep submergence, and that on the top of the hood an air-valve had been laced. That was all that was added and all that was required in that particular case. It might be that if deep submergence were adopted on the ype shown in *Figs. 13* (p. 486), an air-valve might also be necessary, but he low entrance-velocity obviated the necessity for deep submergence.

The reason for requiring the air-valve in the design shown in *Fig. 12* w that under certain conditions there was a large volume of air in the pipe ; the water rose in the hood it compressed the air, and the amount of con pression required was such that the water outside the hood would requi to rise some distance above the crest before any water at all went ove whereas with the air-valve the pressure in the whole siphon system w kept at only a fraction above atmospheric, and as the water rose insid the hood the excess air was blown out. Only when the water began trickle over and produce a lowering of the pressure did the air-valve close

Mr. Morgan made some interesting remarks on the history of the vario formulas, an aspect of the case which the Author had not investigated any great extent, and Mr. Morgan also referred to the von Kármán-Prand formula, which would be recognized as being far more difficult of man pulation than the Manning formula. It might be of interest to give t figures obtained by applying the Manning formula to the results give by Nikuradse. The sand-grain sizes were in a regular progression rati 1.2.4.8.16, and mean values of n^6 were calculated from the hydraul gradients and diameters as recorded in Nikuradse's Paper. The resul were given in Table XI. A glance at the figures showed that any variatio

TABLE XI.

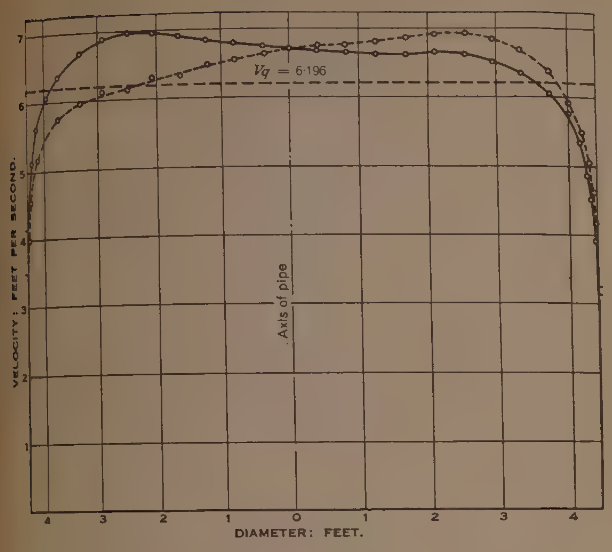
Roughness, K : centimetre . . .	0·01	0·02	0·04	0·08	0·16
Ratio $\frac{k}{k_1}$	1	2	4	8	16
Mean value of n	0·0082	0·0092	0·0102	0·0116	0·0130
Ratio $\frac{n^6}{n_1^6}$	1	2	3·7	8	15·7

from the relation $k \propto n^6$ was far less than would be expected in such a experiment, so that the Author contended that the Manning formula gav at least as good correspondence with the actual conditions of turbulen flow from small pipes to large pipes as the more complicated logarithmi formula given by Nikuradse. It should be mentioned that one sectio of Nikuradse's results was discarded as it was found to be subject through out to an error of calculation in the Paper as published.

*** The Correspondence on the foregoing Paper will be published in the Institution Journal for October 1939.—SEC. INST. C.E.

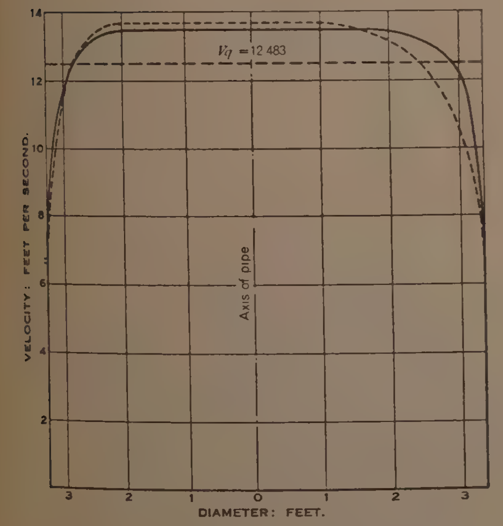
CONSIDERATIONS ON FLOW IN LARGE PIPES, CONDUITS, TUNNELS, BENDS, AND SIPHONS.

FIG: 2(A).



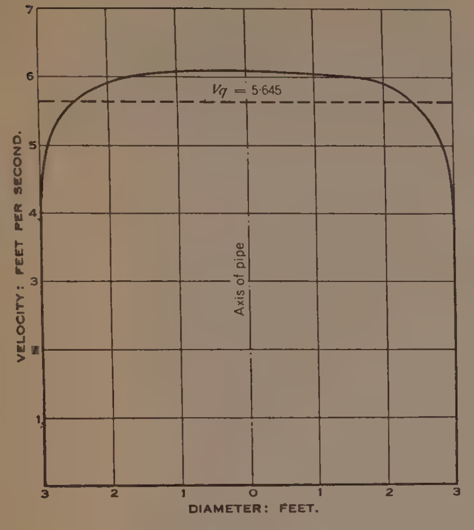
8'.9" DIA. WELDED STEEL PIPE, NEW SMOOTH-COATED.

FIG: 2(B).



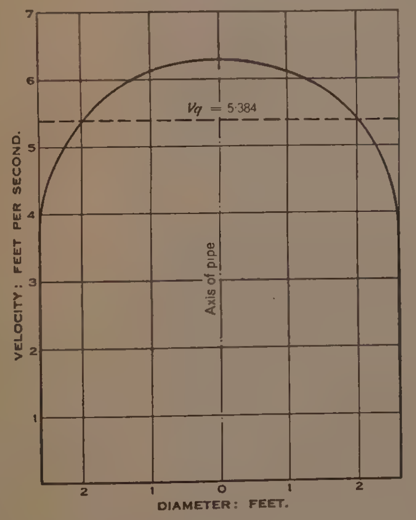
6'.8" DIA. RIVETED STEEL PIPE, NEW SMOOTH-COATED.

FIG: 2(C).



6'.0" DIA. CAST-IRON PIPE, NEW SMOOTH-COATED.

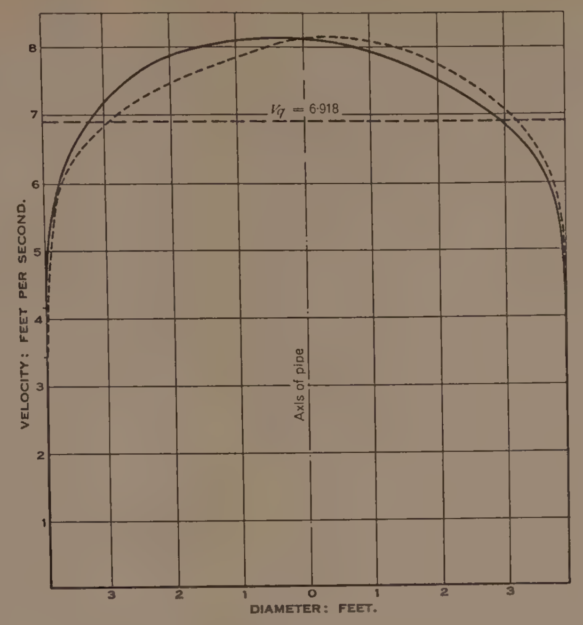
FIG: 2(D).



5'.3" DIA. WELDED STEEL PIPE, ROUGH COATING.

VELOCITY CURVES FOR VARIOUS LARGE PIPE-LINES.

FIG: 2(E).



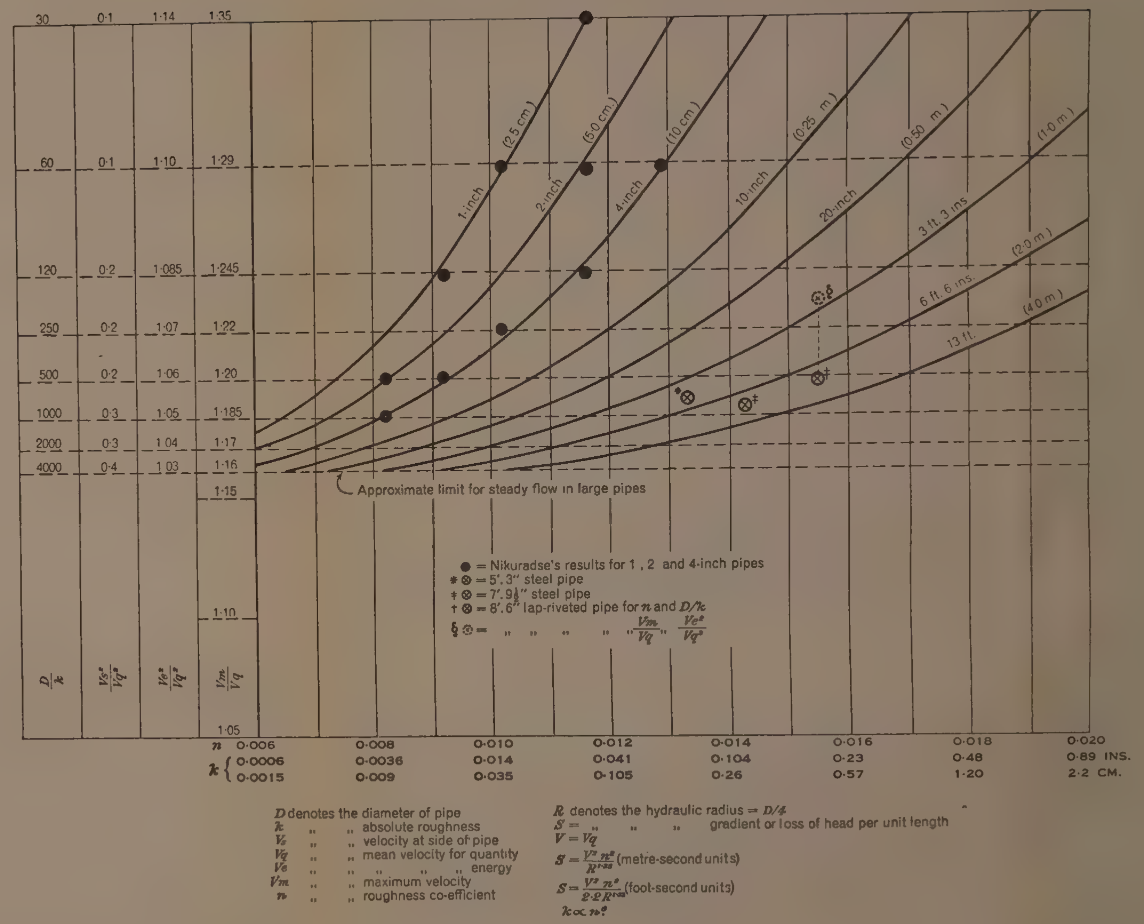
7'.9½" DIA. WELDED STEEL PIPE, ROUGH COATING.

FIG: 3.

8'.9" DIA. PIPE INDICATING SPIRAL FLOW.

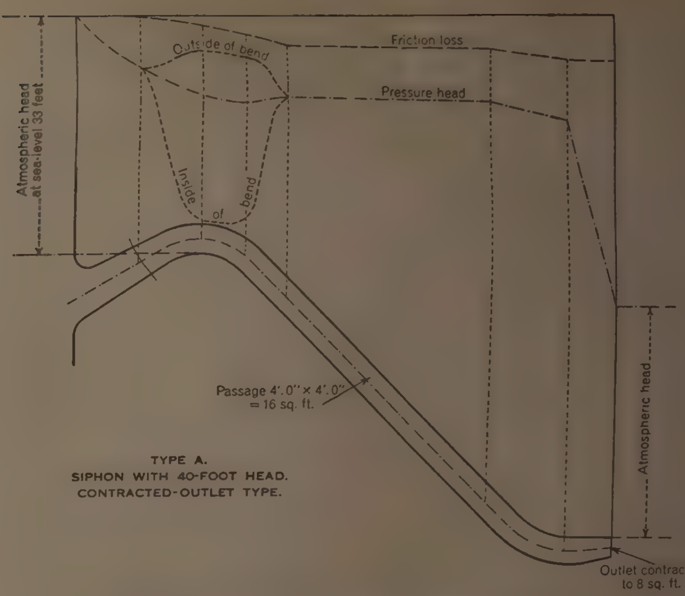
CONTOUR-DIAGRAM OF VELOCITIES.

FIG: 4.



APPROXIMATE SIMILARITY CHART FOR FLOW IN PIPES WITH ROUGHNESS-EFFECT FULLY DEVELOPED.

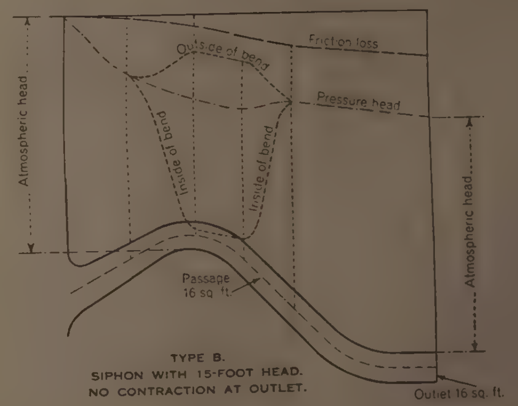
FIG: 9.



TYPE A.
SIPHON WITH 40-FOOT HEAD.
CONTRACTED-OUTLET TYPE.

DEVELOPED DIAGRAM SHOWING PRESSURE-HEAD AND FRICTION LOSS.

FIG: 10.



TYPE B.
SIPHON WITH 15-FOOT HEAD.
NO CONTRACTION AT OUTLET.

DEVELOPED DIAGRAM SHOWING PRESSURE-HEAD AND FRICTION LOSS.

JAMES WILLIAMSON.

WILLIAM CLOWES & SONS, LIMITED: LONDON.

ORDINARY MEETING.

21 March, 1939.

WILLIAM JAMES EAMES BINNIE, M.A., President, in the Chair.

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

Paper No. 5211.

"Reconstruction of Aldgate East Station." †

By JOHN HARLEY HARLEY-MASON, M. Inst. C.E.

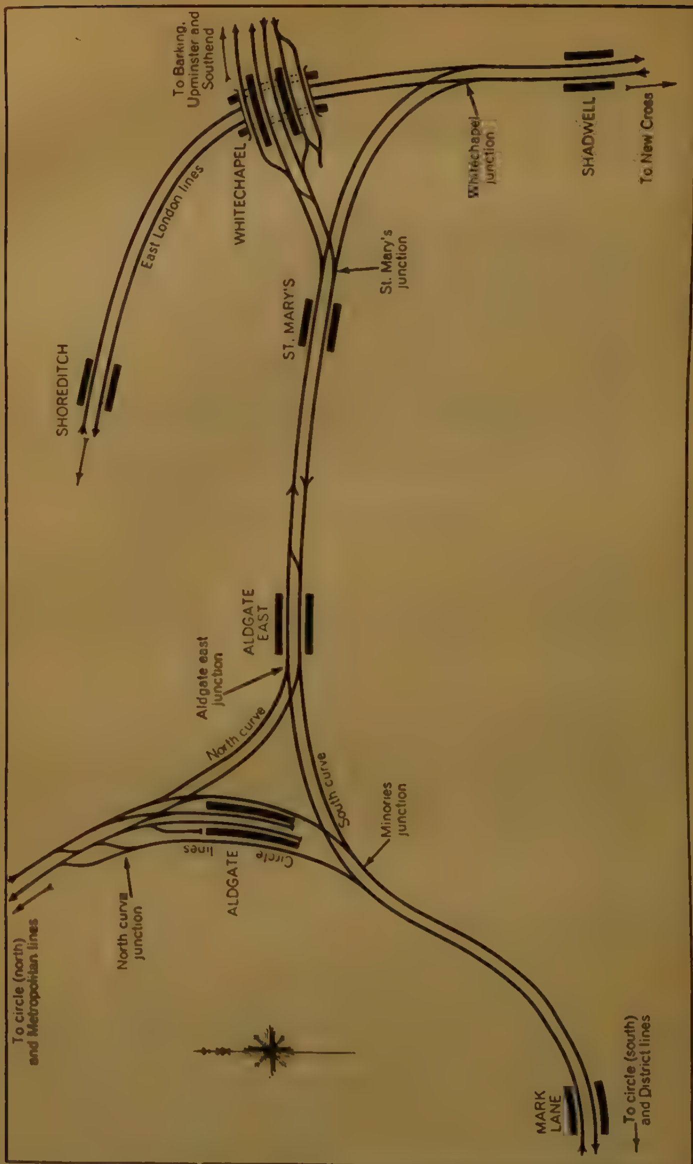
TABLE OF CONTENTS.

HISTORICAL.

IN 1863, the House of Lords Committee recommended the completion of the Inner Circle railway, and, in the following year, the Metropolitan Railway was authorized to extend from Moorgate to Tower hill, the Metropolitan and District Railway Company being formed to build the southern half of the Circle line from Tower hill to South Kensington, in conjunction with the construction of the Victoria embankment, then being commenced. Rival interests delayed the completion until 1884, by which time not only had the Inner Circle been completed but also the triangle of junctions at Aldgate giving access to Aldgate East and St. Mary's, and

† Correspondence on this Paper can be accepted until the 15th July, 1939.—SEC. INST. C.E.

Fig. 1.



OLD LAYOUT IN THE VICINITY OF ALDGATE EAST STATION.

then making a junction with the East London line, which had been buil[t] in 1876. At St. Mary's was a spur-line into an engine-shed and car-siding[s] north of the main road, where Whitechapel station now stands.

This spur has now become the main line through West Ham and Eas[t] Ham to Barking, Upminster and Southend, the traffic on which has grow[n]

to such an extent as to make the present reconstruction at Aldgate East essential, as this was the bottle neck preventing further expansion. *Fig.1* shows the old layout in this vicinity.

The Aims of the Work.

The problem to be dealt with was not only to increase the capacity of the train-service eastward, but to relieve the congestion of pedestrians on the overcrowded street-area between Aldgate pump and Whitechapel church, and also to deal adequately with the large number of persons using Aldgate East station itself.

The following points required a remedy:

(1) The short sides of the triangle referred to only accommodated six-car trains between the fouling points at the apices, and the absence of overlaps required by modern signalling was a source of delay.

(2) The Aldgate East platforms were only long enough for six-car trains, although eight-car trains were running, and the access was at one end of the platforms, causing congestion.

(3) The Aldgate southbound Circle platform was only of six-car length, and this was too near the fouling point with the east-bound District line track.

(4) Whilst Aldgate station was well placed near the Minories, Houndsditch and Aldgate traffic-centre, Aldgate East was too near to be a satisfactory station on its own, and it was not at a natural traffic-centre.

(5) The length of High street between Aldgate East and St. Mary's was unprovided with railway facilities, although it included the large traffic point at Gardiner's corner formed by the junction of Commercial road, Whitechapel High street, etc.

(6) St. Mary's station was near to Whitechapel station and not at a traffic-centre.

In the last 20 years many schemes have been put forward, each to deal with one or two of the difficulties, but they always increased those difficulties that remained; for example, fly-under schemes at great expense would increase the train-service from the City to east of Whitechapel but the local Aldgate East passengers would be worse off than before.

Only a bold scheme could solve all these problems, and such a scheme (shown on the general plan, Fig. 2, Plate 1), has been carried out in the following manner:

A new double-ended Aldgate East station has been built, east of the old station, which was scrapped when the new one was completed. A new south curve has been built outside the triangle, thus lengthening all three sides, and the old south-curve tunnel has been abandoned.

This scheme deals with all the six points before referred to.

(1) Eight-car trains can now run with ample overlap.
(2) There are now eight-car platforms at Aldgate East with acc from each end on both sides of the street.
(3) Aldgate southbound platform has been lengthened and the layo improved.
(4) and (5) The ample street entrances east and west of Gardiner corner not only collect passengers without the need f crossing the crowded main road, but also provide pedestria subways.
(6) The new entrances at the east end, 310 yards east of the o Aldgate East booking hall, allowed St. Mary's station to l closed.

To the above improvements has been added a large road-transpo station, for the London Passenger Transport Board's omnibuses, moto coaches, and trolley 'buses, formed by covering in the south junction the triangle at the Minories, which will provide a turning point for t trolley 'buses now replacing the trams at that point.

The Obstacles to the Work.

The above scheme having been adopted, it was necessary to revie the complications in carrying out the work.

(*a*) It was imperative to carry out the work without to interrupti the railway traffic. Occupation of the line was limited to fro 1.0 a.m. to 5.0 a.m. on weeknights and from 1.0 a.m. to 7.0 a.m on Saturday nights, this occupation being frequently inte rupted by works-trains. The existing Aldgate East statio had, of course, to continue functioning until the new statio was ready to be opened.

(*b*) It was also essential to maintain uninterrupted service of th tramways (the conduit-type tracks for which are place approximately over the centre of the tunnel in Whitechap High street, with a very complicated system of junctio at the point where Commercial street and Leman street joi Whitechapel High street). It was possible at times to sto some of the all-night trams short of the work for a ver limited period.

(*c*) Conference with the local authorities and the police disclosed t fact that eastbound road traffic could not be diverted fro Aldgate and Whitechapel High streets, but that the wes bound traffic could be diverted from Whitechapel High stre via Church lane, Great Alie street and Mansell street, althoug it could not be diverted from Aldgate High street. Also t road-junctions at Commercial street and Middlesex stre had to be kept free of obstruction to cross traffic at all time

(*d*) Surveys and records disclosed that sewers existed on both sides of the railway tunnel, and crossed under, just below the tracks, at Commercial street, Middlesex street, and Little Somerset street.

The problem presented by the various public supply installations was also a very large one. The Gas, Light and Coke Company had large 36-inch diameter mains running along with the tunnel for the full length of the works, and two 48-inch diameter gas-mains crossing over the railway in a special pipe-bay at Commercial street.

The Commercial Gas Company had also mains up to 18 inches in diameter on either side of the tunnels and also crossing at Commercial street pipe-bay.

The Metropolitan Water Board had mains up to 20 inches in diameter and the London Hydraulic Power Company had 6-inch diameter mains, all following much the same routes.

The General Post Office had many cables under both pavements with frequent crossings over the railway.

The Stepney Borough Council Electricity Undertaking, the Tramways high-tension and low-tension distributions, the London Power Company, the Charing Cross (Central London) Electricity Supply Company, and the City of London Electricity Company, all had high- and low-tension cables in large banks of ducts running both with the tunnels and crossing the railway.

(*e*) The properties likely to be affected were generally of an aged and somewhat dilapidated nature, although in nearly every case they were vested with heavy trade interests, and it was obviously desirable to interfere with these trades as little as possible. The properties on the south side of Aldgate High street are primarily a wholesale kosher-butchers' market at nights and occupied by retail tradesmen by day. This, together with the impossibility of diverting road-traffic at this point, made it necessary to carry out the work on the new south curve covered-way below the surface in headings.

From these facts it will be understood that the method of carrying out the various operations was the dominating factor in the details of design.

The Models.

The portion of the works contemplated at the West ticket-hall near Commercial road, where no hoardings could be allowed, was so complicated by the large number and size of sewers, pipes, and mains, and by the existing large tram-track layout, which could not be interfered with, that

resort had to be made to two models (*Fig. 3* and *Fig. 4*, facing pp. 508, 509) built up to a scale of ¼ inch to the foot from the surveys and from information collected. One model (*Fig. 3*) showed everything existing before carrying out the works, every known feature, both above and below ground, being faithfully reproduced. The second model (*Fig. 4*) was of the same area, but embodied the new works and in it the diversion of all sewers, mains, pipes, etc., were worked out, this proving easier than any attempts to do so by drawings.

The models proved of enormous value, not only in demonstrating the possible diversions, and enabling the various Undertakers and Authorities to satisfy themselves of the best solution of their part of the problem, but also in enabling the Contractors tendering to visualize the actual conditions. They have also been of great use constantly throughout the works in showing easily the position and proximity of various features as the works progressed.

Contracts.

Early in 1936 the first contract was let for all that part of the work east of the old station and certain other works on the north side of the old station, and in February 1937, the second contract was let for the reconstruction of the old station-structure, the new south-curve covered-way, and the work at Minories junction.

Street Occupations.

The street occupations which were arranged for the works will be referred to as "A" for work north of the existing tunnel, "B" for all work south of the tunnel, and "C" for work along the centre of the road over the railway, these occupations referring to street-traffic only. The presence of sewers and mains parallel to the railway for the whole length of the works made this longitudinal working necessary, and as the main road carried one-way traffic only, long lengths of hoarding could be erected and the works expedited.

It was originally intended to complete occupation "B" before commencing occupation "C" (the centre of the road), but various considerations modified this, and occupation "C" was put in hand earlier, and closely followed the work going on under occupation "B." By this means a contemplated programme of 3 years was reduced to an actual period of 2 years and 7 months.

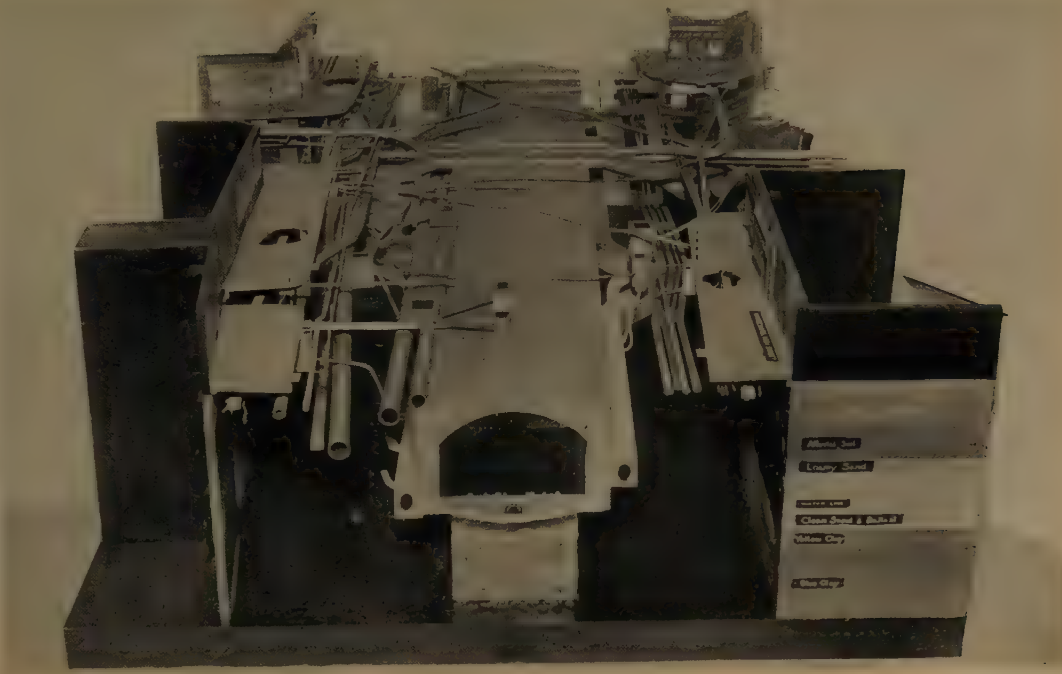
Rail-Traffic Arrangements.

The work affected the railways in two stages:

During the first stage the track layout was practically unaltered and all trains used the old platforms.

The first stage terminated at Changeover I during the last week-end of

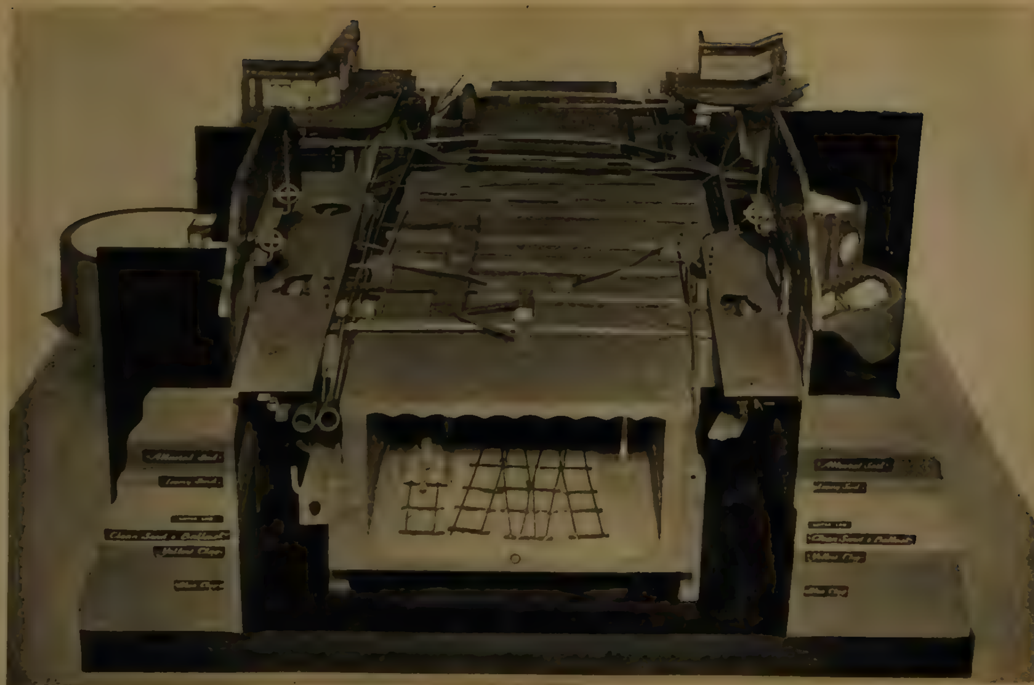
Fig. 3.



MODEL OF NEW WEST TICKET-HALL SITE BEFORE COMMENCEMENT OF OPERATIONS

5211.

Fig. 4.



MODEL OF WEST TICKET-HALL SITE AT COMPLETION OF OPERATIONS.

October 1938, when the new station was opened and the old platforms removed.

During the second stage, lasting for 4 weeks, the new station was in operation, but with the tracks in the old south-curve tunnel still in use. During this stage (with the old platforms cleared and the tracks lowered to the new level at Changeover I), it was possible to lay in the new track through the new south-curve and complete the junctions at the Minories and just west of the new station, together with the final signal-equipment. At Changeover II, on the night of Saturday, 26 November, the final layout was brought into use.

A completely new signal-cabin has been built over the Circle lines, just north of Aldgate station, which will control the whole layout in place of the obsolete cabins at Aldgate East and Aldgate stations.

The Eastern Approach.

The description of the works and the methods employed will commence at the extreme east end, and the contract comprising the new Aldgate East station will be dealt with first, together with the alterations to the old tunnel east thereof, and also the new bellmouth covered-way between the new and old station-sites.

A reference to Fig. 5, Plate 1, which is a cross section at the new east ticket-hall and entrance-subways, shows that, in order to obtain headroom for the ticket-halls below the street, it was necessary throughout the length of the new station to lower the railway-tracks approximately 7 feet. East of the new station an up-gradient of 1 in 40 to the old track-level was provided in the old tunnel by thickening and underpinning the walls and reconstructing the invert at lower levels (*Fig. 6*, p. 510). West of the new station a gradient of 1 in 52 up to the old level was provided, through the old station-site, by lowering the invert (Figs. 11, Plate 1).

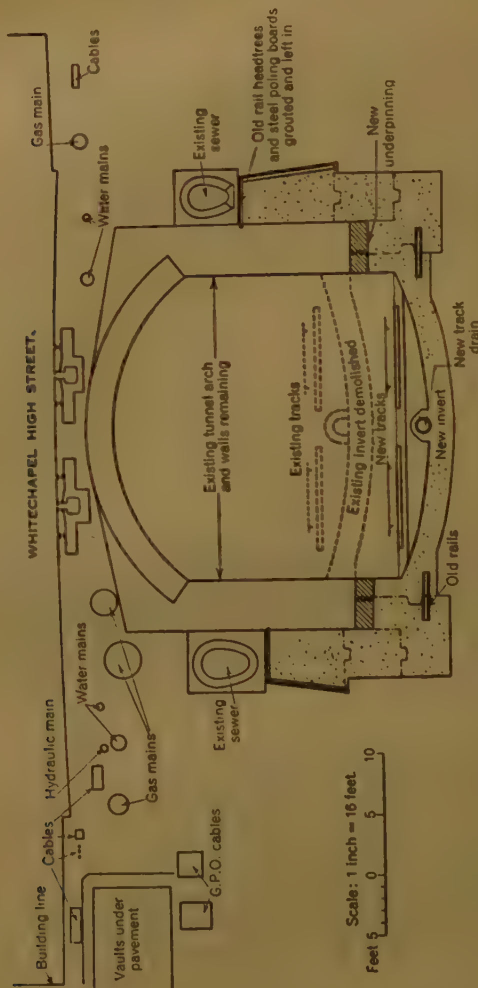
Except for an initial lowering of the tracks, of from 6 inches to 15 inches, by taking out some of the ballast throughout the site, to give extra head-room for new girders and demolition-shields, etc., the tracks remained throughout in their original position, being lowered in one operation to their final level during the changeover.

The work indicated on *Fig. 6* in the old tunnel, east of the new station, was carried out by driving headings, from openings left in the new east headwalls, along the back of the old walls and just below the sewer. From these headings the rear part of the old walls was underpinned in 4-foot lengths, leaving old rails projecting into the earth to form a bond with the inner portion and the new invert, which were constructed at a later stage.

From pits at the far ends of the headings a cross heading was driven under the old tunnel-invert, and a certain amount of the new invert-work and trestling of the tracks, as described later, was carried out from

this heading. As this proved very slow, however, owing to the difficulty o handling material through the headings, the new invert work on the ne station caught up, and most of the invert work of the old tunnel was deal with by working on a "face" from the tunnel-end. As the new invert

Fig. 6.



CROSS SECTION EAST OF NEW STATION (SECTION AA, FIGS. 2, PLATE 1).

was constructed and the old invert was taken out, the inner 18 inche of underpinning was completed under the old tunnel-walls.

The headings were then filled with concrete and the sewers underpinned The old rail head-trees and steel poling-boards were left in, having bee provided for this purpose when the heading was constructed.

The whole was grouted up with cement-grout to fill the many voids found below the old sewers and between the sewers and the tunnel-walls.

The New Station-Site.

North Side (Occupation "A").

At the new station-site a reference to Figs. 5, 7, and 9, Plate 1, will show the relation of the new structure to the old tunnel.

Dealing first with the platform-section (Figs. 7 and Fig. 8, Plate 1), as soon as the north side of the road was occupied, a series of pits approximately 25 feet long were sunk on the centre-lines of the main stanchions situated on the centre of the new platforms, these being generally at 52-foot 6-inch centres. The pits, extending from the back of the old tunnel-wall to the extreme outside of the new sewers, were first excavated to the level of the bottom of the sewers, the old sewers being undisturbed and water mains, etc., suspended overhead.

From this level, which was just above the normal water-level in the soil, and always kept above a line drawn downwards at 45 degrees from the foundations of the nearest buildings, steel sheet-piling was driven on the line of the outside of the main retaining walls to some feet below the lowest excavation. This sheet-piling was left in, the Contractors being allowed to use any serviceable secondhand-piles of suitably strong section. Excavation then proceeded to the lowest level, but on the inner face the excavation was carried down on the line of the outer side of the sewer, timber runners being used which extended to the top of the sewer when fully driven.

This precaution was taken as the sewer was an old brick barrel which required careful handling. Extra struts and heavier walings were placed opposite the springing of the tunnel-arch to prevent the latter spreading. Having bottomed up the main excavation, a 12-inch layer of concrete was placed to seal the bottom. Then headings were driven under the old sewer and old tunnel wall for the insertion of the large grillage-foundations for the main stanchions. As the lower tiers of grillages were concreted, the old tunnel wall was underpinned in the headings with concrete. At this stage it was thought desirable to make further provision for the horizontal thrust from the arch. It was realized that the tunnel would ultimately be completely stripped on both sides, and perched up on a 7-foot high dumpling of indifferent clay and ballast (which would be totally de-watered), and, at the same time, the arch would be fully loaded. To stiffen the tunnel and provide for the horizontal thrust at the springing, the temporary steel-struts, seen in Fig. 8, Plate 1, were inserted and wedged up again a continuous waling opposite the springing point and against a vertical timber soldier on the outer trench-face.

These struts had short angles riveted on to them to form anchors within the new concrete retaining-walls when cast, automatically

transferring the thrust of the arch to the heavy retaining-walls without re-timbering.

This allowed the outer ends of the struts to be burnt off to complete the asphalt damp-course and the back-filling, and, at a later stage, after the arch was demolished, the inner portion of the strut was burnt off and the face of the wall made good.

By this means, any serious spreading of the arch, which might have occurred had the usual method of successive changing of timbers been employed, was prevented.

When these struts had been inserted, the new concrete walls were cast up to the roof-level. The intervening pits were then excavated and dealt with in a similar manner. One of the reasons for splitting up the walls into about 25-foot lengths was to allow for initial contraction, future expansion and contraction, and any slight variation in settlement as the walls became fully loaded. Being asphalted at the rear and tiled in the front, it was very desirable to avoid the large cracks that might have occurred had these very long walls been cast in one continuous length. A special vertical V-joint was designed at the ends of these 25-foot lengths, wrought shuttering was employed, and the surface of one length of wall was given a double coat of shutter-compound before casting the next section of concrete against it, to prevent actual adhesion. This method was adopted throughout where practicable, and it is satisfactory to note that in the length of the south wall, nearly 1,700 feet long, there is no sign of anything more than hair-cracks at some of the joints. Having completed the wall, the new sewer was put in hand and, as soon as the full length was completed, the house connexions were changed over and the old sewer broken out at the stanchion-positions. Stanchions and main longitudinal girders were then erected, being brought to the site by road and lowered into position at nights to avoid interference with tramway-traffic. It is here that the earlier statement, that the design was sub-ordinated to the site-conditions, is well illustrated, the steelwork being positioned so as not to interfere with the old tunnel. The outer bay of roof-steel was then inserted, at a level which avoided diverting the existing mains, the roof completed, and the road made good again. A small temporary strip of timber road-decking was inserted over the longitudinal girder, to allow subsequent access for putting in the centre span of roof-girders, the road-filling being held up by a temporary concrete ballast-wall.

At the same time, the remainder of the old sewer was demolished and the ground excavated to a line coinciding with the outer face of the old tunnel-wall, and a further strip of concrete-invert formed. Extra timber struts were also provided at this stage between the bottom of the old and new walls to prevent any possible lateral movement. Wherever it was found necessary to break up the new tunnel-invert in the transverse direction, old bull-headed rails 4 feet long were inserted across the joint to bond the whole together.

South Side (Occupation "B").

After making good the road, the road-traffic was transferred to the north side, and the whole of the south side occupied (already described as occupation "B"). The work south of the old tunnel was then completed in a similar manner to that on the north side and the road made good.

Occupations "B" and "C."

When sufficient of this work was completed, the demolition of the old tunnel-arch was begun, hoardings being extended to the north side of the tramway-tracks (already described as occupation "C"). As the tramway-tracks had to be maintained in uninterrupted use throughout, the centre girders were shaped in "fish-bellied" form (Figs. 7, Plate 1) and were kept at about 10-foot centres to allow for support of the tramways over the gaps in the tunnel-roof, while each panel of old tunnel was demolished and the new steelwork inserted.

The tramways were carried across the gaps on 9-inch by 7-inch service rolled-steel joists, inserted just below the tramway-rails and, as the new roof-girders were erected, the joists were packed off them, timber sheeting being laid over the filler-joists during the day to protect the railway below.

The Tunnel-Demolition Shields.

As only about 4 hours each night were available on the railway, it was thought desirable to provide shields, fitted to the soffit of the old arch, to enable demolition to go on uninterruptedly during the full 24 hours each day. These shields, shown in Figs. 10, Plate 2, were 14 feet long and consisted of ribs of 5-inch by 4½-inch rolled steel joist at 3-foot 6-inch centres bent to the shape of the arch soffit.

Between the ribs, hardwood purlins were provided on which corrugated-iron sheets of extra heavy gauge were fixed, of a curvature to fit snugly against the brick-arch.

The whole of this shield was supported on brackets which were bolted to the tunnel-walls just below the arch-springing. The brackets were provided with jacking screws supporting double-flanged rollers, on which the shield proper was mounted, by means of a steel angle-track permanently fixed to the lower ends of the steel-ribs, with the upright leg of this angle running between the roller-flanges. The angles were made several feet longer than the shield to give a "lead" on to the next set of brackets when the shield was moved.

The brackets on the tunnel-walls were positioned to coincide with the steel ribs when the shield was stationary, and it was held tight against the arch-soffit by means of packings inserted between the feet of the ribs and the tops of the wall-brackets.

On the completion of any one length of arch-demolition, a girder and

its panel of filler-joists were brought to the site by road, and off-loaded and suspended from an erecting pole placed alongside the hole on the south side of the tramway-tracks. This girder was slung with its north end heavy, so that it could be paid out into the tunnel in a nearly vertical position.

On the previous night, an extra set of wall-brackets was fixed ready for the demolition-shield in its next position. Immediately the last train was clear of the job, the packings between the ribs of the shield and the wall-brackets were taken out and the whole shield was lowered to clear the tunnel-soffit by slackening the screws. The shield was then drawn forward by ropes, on the rollers of the brackets, to its position ready for demolishing the next length of arch.

When the shield was moved, the end of the fish-bellied main girder was lowered into the tunnel and picked up from the road above by the boom of a mobile crane passed through on the north side of the tramway-tracks, the decking left in after occupation "A" being taken up for this purpose. The girder was then raised level and eased over to its final position with its ends supported on the two longitudinal girders previously erected on the outside of the old tunnel-walls. The bay of filler-joists was then erected, and on succeeding nights, the shuttering was erected and the roof concreted and asphalted, the tramways re-supported on the new roof, and the whole road made good.

The New Ticket-Halls.

At the east and west ticket-halls (Fig. 5, Plate 1) and at the new bellmouth-tunnel, the order of work proceeded on similar lines.

Referring to Fig. 5, Plate 1, it will be seen that there are some significant variations in the design adopted. In order to avoid having to lower the tracks unduly and to avoid any more steps than actually necessary between the platforms and road-pavements, the construction depth of the ticket-hall roof and floor was reduced to a minimum. It will be seen that the roof-girders have been designed to be inserted under the tramway-conduits with the minimum possible clearance for asphalting. They have also been designed in three pieces, the outer portions cantilevering out over stanchions carried up from the platforms, and anchored down into the mass of the new retaining walls by channel-steel anchors, the centre portion being a simple span, site-jointed to the two ends. The floor-joists have been treated in a similar manner, cantilevering out to a point to clear the loading gauge with the railway-tracks at their original levels, and pinned at the outer ends into the main walls.

The centre portion of the ticket-hall floor is suspended from the roof at the centre by means of $2\frac{1}{2}$-inch diameter round mild-steel hangers. Owing to the tracks being kept at the high level throughout until the changeover to the new station, provision for rapid insertion of the centre portion of the ticket-hall floor was made.

The Bellmouth: New and Old Stations.

At the bellmouth-tunnel section, between the old and new stations, the final track layout prevented the use of any stanchions between the main walls, so that the main girders formed a clear span. Owing to erection difficulties, and occupation of the public road being confined to one side at a time, it was necessary to design these girders in three pieces with site-joints. The north ends of these girders were erected during occupation "A," the inner ends resting temporarily on a bearing formed in a pocket cut in the old tunnel-wall. The south and centre portions were erected during occupation "C," the centre-piece being raised from the railway below into a chase cut in the tunnel-arch, and the south end being inserted from the roadway. As soon as the site-joints were completed, wedges were driven under the south end of the girders until the temporary bearing on the north wall just showed a hair-crack. The intervening panel of arch between these main girders was demolished on shields as described above, but with no provision for travelling the shields along the tunnel.

A point of interest in the design here was the use of pre-cast concrete jack-arch centerings, which were used to avoid suspended timber centres over the running tracks. These centres are of a standard design and were used also to a great extent in the new south-curve tunnel, and the new deck over the Minories junction.

Drainage-Sump.

On the cross section, Fig. 5, Plate 1, will be noted the permanent pumping-plant and large sump to which is led all water from the dip in the tunnel, caused by the lowering at the new station. Considering that there is no tanking of the new station structure below platform-levels, and elsewhere just below main girder seatings, and that the normal water-level in the ground is about 5 feet above new rail-level at this point, it is satisfactory to note that the quantity of water dealt with at the new sump remains consistently below 500 gallons per hour, a good proportion of which comes from the old tunnel east of the new station.

Pipe-Bay over Station.

The work at the old pipe-bay at the point where Commercial street, Commercial road, and Leman street join Whitechapel High street (Figs. 2 and 9, Plate 1) is worthy of special mention here. The old pipe-bay, situated at a point in the roads where the large volume of traffic precluded any hoardings, and under the complicated system of tramway-crossings and junctions, presented a difficult problem.

The old construction consisted of four shallow girders, let into the tunnel-roof, between which were bedded two 48-inch diameter gas-mains.

an 18-inch diameter gas-main, and a 20-inch diameter water-main, nor of which could be taken out of service except for very brief period. Eventually it was decided to leave this pipe-bay intact, together with it pipes and the tramway-tracks, only altering the pipes at the north an south ends enough to give room for the platform roof-structure at thes points.

It can be seen by referring to Figs. 9, Plate 1, that the old girders wer taken off their bearings on the old tunnel-wall on to new cantilever anchored down into the new retaining walls at the back of the platforms.

The work here was put in hand on similar lines and at the same tim as the work east and west of this point, but, of course, was seriousl complicated by having to be carried out entirely in headings driven from the hoarded areas on either side.

The main walls were constructed, the sewers diverted, and the com plicated diversions of mains put in hand, all in similar headings.

Besides the sewers running longitudinally on both tunnel-walls whic were to be set back, there was the further complication of junctions wit sewers north and south from Commercial street and Leman street, and siphon under the old railway-tunnel.

Before proceeding too far with the new wall on the north side, the ne siphon shown on Figs. 2 and 9, Plate 1, was constructed, working entirel from a heading on the north side. The top half of the shaft of the sipho on the south side was not dealt with at this stage, but the new penstoc chamber connexion to the London County Council No. 2 low-level sewe was constructed to enable the sewage to be dealt with, as the new chamber and connexions were under construction on the north side.

As will be gathered from the various plans and sections, numerous an complicated pipes and mains diversions had to be undertaken in th roads to make room for the new structure. These were at their maximum in this particular area, and the alteration of the 48-inch gas-mains a the south side of the pipe-bay, being mostly in heading, was a ver awkward piece of work, and a special tribute is due to the Gas, Light an Coke Company's Engineers and their Contractors for the way in whic the work was carried out.

The introduction of the new steelwork at the north and south ends o the Commercial street pipe-bay is also of some interest. As soon as th work on the main walls, sewers, and diversions was sufficiently advance headings were driven along the rear of the top of the old tunnel-wal and, from these headings, pits were sunk for grillages of the new stanchion situated at each corner of the old pipe-bay. The grillages were concrete in, stanchions threaded into the headings and down the pits, and the shor longitudinal girders drawn along the headings into their positions on to of the stanchions. The four cantilever-girders were similarly put int position, the outer ends bolted to anchors in the main walls and the inne ends jointed up to the longitudinal girders and to the cantilever-bracket

on the far side of these girders, which had been fixed to the latter prior to erection.

As will be seen by reference to Figs. 9, Plate 1, the outer ends of these brackets, when in position, came under the bearings at the ends of the old pipe-bay girders. Before cutting away the existing blue-brick seatings of the latter, temporary short steel stanchions were introduced in chases, cut in the inner face of the tunnel-wall, under each old girder. When the new steelwork was all connected up, fitted steel folding wedges were driven home between the tops of the cantilever-brackets and bearing-plates of the old girders, and the temporary stanchions, and the remainder of the brick seatings, were then removed.

The new roof was then completed, the remainder of the pipe-diversions carried out, and the void under the road surface back-filled with weak ballast-concrete.

Supporting of Road.

In carrying out such work, where large surfaces of road had to be left in, the principle adopted was as follows: headings were driven first, just below the existing road-surface concrete on the lines of the new walls, sewers, girders, pipes, etc., sinking from the bottom of the headings to foundation-levels, after which walls were concreted, or steel-erected, as far as the circumstances would permit. The road surface was then temporarily picked up by brick piers built up off the finished work, dumplings between headings were taken out and the road surface supported on steel joist head-trees off the brick piers.

As the new roof was completed, the steel head-trees were transferred to new brick piers built on the roof, the older piers demolished, and so on. When stuffing the space between the road surface and the roof, these piers and steel joists were left in where they could not safely be taken out.

The Old Station-Site.

At the old station-site (Figs. 11, Plate 1) certain works on the north side were put in hand during road-occupation "A," under the first contract, as a matter of convenience.

The London County Council sewer was diverted from its old position, just below the surface of the old eastbound platform to the rear of the tunnel-wall, being built in cast-iron segments, lined with concrete and blue brick, and driven from the excavations within the road-hoardings just east of the old station.

At the same time the north wall was underpinned with concrete and brickwork carried well down into blue clay, in order to provide foundations for the greatly increased loads that would be developed from the large span of the new roof-girders.

At the eastern end, where these girders have new seatings formed in

chases cut in the old wall, the full width of the wall was underpinned an a very much wider base provided at the lower level.

This was done by driving a heading between the back of the old wa and the new sewer and sinking in short lengths, driving headings right und the old wall to the full " toe " width, and then filling back with concre and brickwork underpinning, the heading subsequently being filled wit concrete, thus adding thickness to the old wall. Further west, where th track-layout permitted the new roof-girders to be supported on stanchion the wall-underpinning and lowering of invert was carried out, in the secon contract, from inside the railway-structure, working under temporar platform-surfaces. At this time the existing concrete-and-asphalt pla forms of the old station were broken out at night, their supporting wal trimmed down about 18 inches and a temporary timber-surface provide This enabled the tracks throughout the old station to be lowered by takin out some of the ballast to an initial depth of 15 inches, to give reasonab construction depth for the large new roof-girders (Figs. 11, Plate 1).

As it was impossible to lower the platforms immediately 15 inche without the track being lowered as well, this was done in three stages 5 inches at a time, three sets of packings of this thickness being inserte under the temporary platform-surfacing.

On a Saturday night the whole of the first series of 5-inch packing were removed, and during the succeeding week, at nights, the track wa lowered to suit.

On successive Saturday nights the second and third series of packing were removed, the tracks following.

As soon as road-occupation " B " commenced, the south side Whitechapel High street was taken over, and the second contract wa also commenced on the south side of the old station.

Increase of Roof Span at Old Station-Site.

Reference to Figs. 2 and 11, Plate 1, shows how, in widening ou the old station, to give the four-track layout at this point, the new sout wall was brought largely under the existing buildings on the south side and large clear-span girders were necessary to support the road and th frontages of these buildings above.

Here again, the early remark made that the design is subordinate t the conditions, is well illustrated. The road-conditions, with the tramway terminus along the centre, and large mains occupying the small amount o road on the north side, indicated that the old roof over the north platform and tracks would have to be left in, but that the columns on the old platforms supporting the roof would have to be removed. Finally, the scheme shown on the section (Figs. 11, Plate 1), was devised.

Large curved girders were designed to fit under the centres of the jack arches, but clear of the brick arches.

The soffit of the girders was shaped to clear the structure-gauge of the

two tracks existing in the station (after the initial 15-inch lowering) and their outer ends to clear the gauge of the four tracks of the final layout at the new levels.

Between these girders were introduced small needling-girders below each end of the old cross girders of the station jack-arched roof, and fitted steel folding wedges, between the needle-girders and the old girders, were employed to take the roof-load off the old columns.

These old columns were at 25-foot centres and supported longitudinal girders, each of which in turn supported two jack-arches, that is, one old cross girder came on the columns and the next came midway along the old longitudinal girder.

As the shape of the new main girders necessitated cutting through the old longitudinal girders on the south side, it was necessary to provide temporary support for the old jack-arch girders midway between the existing columns. This was done by providing the temporary stanchions and grillages shown on the cross section (Figs. 11, Plate 1). Two sets of these were provided so that one could be moved forward each week as the new girders were erected and wedged up, thus enabling the work to proceed at the rate of one main girder each Saturday night.

As there were twenty-one of these girders, this was necessary to keep up a fair rate of progress. Before erecting the main girders a considerable amount of work had to be completed on the south side.

Work under Business Premises.

Prior to commencing work, arrangements were made with the various property-occupiers to lease only such parts of the fronts of their basements as would be affected by the works, to ensure that their business activities would not be seriously interfered with.

In two places only were the London Passenger Transport Board forced to occupy the ground floor, and these places were used as the sole points of access from the street.

It was known from old records that the frontages of these buildings had been underpinned to a certain extent when the railway was first built. This fact was taken advantage of in the scheme. When the underpinning of the party-walls, in the rear of the new wall and sewer, was put in hand, the first 4 feet adjacent to the new wall was carried down on to good ballast and kept well above standing-water level, the remainder stepping up at 45 degrees (Figs. 11, Plate 1). This first block of underpinning was carried out about 3 feet wider each side of the party-wall foundation and on top of this piers were built to within 3 feet of the underside of the ground-floors.

Steel service joists were then placed each side of the old party-wall, their rear ends supported on the piers on the new underpinning and their forward ends carried into the old brick frontage-walls on the original underpinning.

Small steel needles were then built into the party-walls protruding each side over the service joists, and these needles were wedged up tight.

The old party-walls below the joists were then demolished for the width of the new retaining wall and sewer, allowing excavation to proceed for the full length of the wall down to a point just above the level of the bottom of the new party-wall underpinning.

From this level a continuous row of borehole-type concrete piles was cast along the back of the excavation down to well below lowest foundation level. This system of piling was adopted, as driven steel sheet-piling was out of the question in these old buildings, from considerations of vibration, noise, and curtailed headroom.

By this means excavation proceeded safely without undue anxiety as to the water-logged sandy gravel under the building foundations in the rear of the works. During excavation it was found that, except in two places when the piles had run out of plumb, the adjacent piles had coalesced and formed an almost watertight barrier.

Such places were carefully watched for, and dealt with, by cross poling and grouting as excavation went on. From the tops of these piles downwards, excavation was confined to short lengths of from 10 to 15 feet long to a point just above the level of the old front-wall underpinning.

From that level the excavation was carried on in 4-foot lengths and headings driven under the old underpinning to the full extent of the new retaining-wall toe.

As the concrete of this toe was cast, the old underpinning was trimmed square and re-underpinned on to the new retaining wall toe (Figs. 11, Plate 1), and then the new wall was cast up to sewer-level. When the whole wall was brought up to this level, the bearings for the main girders were prepared, a dovetail-shaped chase 3 feet wide being left below each new girder-position in the face of the new wall, from platform-level upwards. The new sewer was then completed and the old one demolished.

In the meantime, excavation proceeded on the south side of the road between the old underpinning and old station retaining-wall, down to the bottom of the latter, a temporary timber pavement being provided for pedestrians in front of the shops, and the various mains slung from the timbers supporting the temporary pavement across the excavation.

Starting from the east end of the old station, preparations were then made for the erection of the new girders. The old station-wall and south part of the roof were demolished in short lengths, and a chase cut through the old underpinning of the buildings corresponding to the girder-chases left in the new wall.

Erection of Girders.

The erection of these girders, some of which were 68 feet long and 30 tons in weight, was a serious problem. The utmost time that could be given on the site was under 6 hours on Saturday nights.

It was first decided that owing to the many obstructions, they must be erected from the railway itself, and that they must be delivered in two pieces, and site-jointed roughly at middle-span. In collaboration with the steelwork sub-contractors, a scheme was eventually evolved whereby each half would travel from the works on a specially adapted railway "crocodile" truck. Figs. 12, Plate 2, show the arrangements. On the truck is placed a circular ball-race consisting of two old rails bent to a circle with large steel balls between. On this, and carrying the half-girder, was a cross-slide, again of old rails and steel balls.

Just below the timber platform-surface, and leading into the chase which had been formed in the walls, a track was also provided consisting of steel joists on their sides and a small turntable-truck running on these joists.

On Saturday night, the works-train, with the two halves of a girder on their special trucks, followed the last passenger train into the station, the north half of the girder on its truck was left on the north track, and the south half shunted on to the south track, opposite the erecting positions. The temporary platform-surface was stripped and the small trucks placed in position. Then the south half of the girder was turned on its "crocodile" truck, and at the same time travelled out on its cross-slide, the end being taken on the small platform truck, and led into its correct position in the chase in the new retaining wall. As the half-girder turned and travelled out into position, the "crocodile" truck was pushed along the railway-track until the centre-end was in its correct position.

The south half being in its right alignment, the north half was dealt with in a similar manner, it having been mounted on its truck at the right height in order to clear the overlapping flange-plates of the girder-joint. Then, still resting on the "crocodile" trucks, the ends were jacked up into line and the joint cover-plates bolted on with turned fitted bolts. Having made the site-joints, the girder was hoisted up into position, the south end being lifted by a winch and a specially made A-frame straddling over the girder-position with its top rising above the road within the hoardings (Figs 13, Plate 2). The north end of the girder was lifted by means of a winch, the bond of which was led over a pulley fixed to the old longitudinal girder of the roof, just to one side of the new girder-centre. The bond was then taken down under the new girder, under two pulleys fixed on to a bracket bolted to the lower flange, and up again to an eye-bracket fixed on the old roof-girder. A guide was provided, clipped to the top flange of the girder being lifted, through which the bond travelled (Figs. 13, Plate 2). This was provided because the centre of gravity of the two lifting points was only very slightly above the centre of gravity of the girder and any tendency for the girder to topple over was thus obviated. As the completed girder was raised to its final position, the ends travelling up the chase left in the walls, the spreader-joists were inserted under the bearings across the chase and the girder landed. The

"crocodile" trucks were returned immediately to the works for the gird for the succeeding Saturday night's erection. The girders were then lin up, the chases filled and the needling girders erected on succeeding night As soon as two girders were completed, the wedges on the needle-girde were driven home to give a predetermined deflexion, and the old column removed.

It was considered advisable to stage a rehearsal of the erection of o of these girders at the works of the makers at Chepstow, before com mencing at the site. The actual conditions at the site were reproduce and the whole operation tried out. This proved of great value, enablin several difficulties to be detected and overcome beforehand, which migh easily have proved very serious on the site during the night's erection.

It was also thought advisable to check the deflexion figures of t girders at the works, and the first two girders made were tested und certain loadings, by being placed back to back, special stirrups being mad to pass round the ends of the two girders and hydraulic jacks introduce between the girders at the loading points. It is interesting to note th the actual deflexions were very close to the calculated deflexions, and th the residual deflexion due to the site-joints taking up was practically nil

It was desirable to be sure of this point and also to deflect the girder to their dead-load strain before taking out the old roof-columns, and befor transferring the existing building-frontages on to the southern ends of th girders. Reference to Figs. 11, Plate 1, will show how the latter detail wa effected by inserting steel joists on either side of the front walls, their end resting on stools built on top of the new girders. Steel needles, built int the walls and wedged up on these joists, then transmitted the building loads on to the new girders.

After this, the old underpinning was cut away and the new railway-roo on the south side constructed.

The road was then refilled, the various mains dealt with, and the roa and pavements re-surfaced.

Inside the basements of the buildings as the wall and sewers wer completed, the party-walls were reinstated and the service-joists an needles removed, the whole redecorated and handed back to the owners.

Lowering the Invert.

The works done under road-occupations "A," "B," and "C," on th old and new station-areas and eastwards thereof having been dealt with the fourth operation (wholly underground), which consisted of the demoli- tion of the old invert and the provision of the new one at a lower level has now to be described. The cross section (Figs. 7, Plate 1), and the track- trestle drawing, Figs. 14, Plate 2, show clearly what was involved.

It will be remembered that in order to maintain full operation of the old station and the railway-tracks, it was necessary to maintain the latter

at about their original levels. Therefore, to carry the tracks at that position while the new invert was constructed, a scheme of trestling was adopted; 1,200 feet of double track being dealt with in this manner.

Two 8-inch by 6-inch rolled steel joists were inserted under each running rail and bolted to the sleepers. These joists were 16 feet long and the joints in each pair were staggered 8 feet and fishplated, so that there was one joint in each pair of joists on top of each of the timber trestles, which were also at 8-feet centres. The trestles consisted of 10-inch by 10-inch pitch-pine head-trees extending transversely under the 8-inch by 6-inch joists of both tracks.

Four 10-inch by 10-inch timber posts, one below each running rail, supported the head-tree off the new invert-concrete as the latter was completed. It was not desirable to provide a timber sill to fix the bottoms of the posts, as it was necessary later, before removing the trestles, to fill in, on the new invert, with ballast up to the underside of the sleepers of the tracks in their final lowered positions. Therefore, the small concrete boxes, shown on the drawings, were formed in the top of the new invert to take the bottom of the trestle-posts; these were filled in with fine, dry, clean sand so that a good bearing was obtained, and so that easy withdrawal of the posts was assured when the trestles were demolished. 11-inch by 4-inch timber cross bracings completed the trestles, and longitudinal bracings between the trestles were also provided. Particular care was taken in the construction of the trestles, so that a perfectly rigid track was maintained, and ease and quickness for removing the trestles was assured at the changeover. The method of removing the old invert and dumpling, the building of the new invert and the insertion of the temporary trestling under railway running-conditions, is of interest. As the new roof of the railway was constructed, large eyebolts had been inserted in the concrete roof-slab, and positioned, one over each running rail, at 30-foot centres longitudinally. The first operation was the insertion of the longitudinal 8-inch by 6-inch rolled steel joists temporarily supported on timber sills under the track-sleepers. One 60-foot length of track was dealt with on each night, and beforehand a 60-foot length of new sleepers complete with chairs, but not rails, was assembled and bolted to its quota of 8-inch by 6-inch joists in the Board's permanent-way yard. This length was mounted on a flat wagon brought to the site at night by a works-train, and positioned over the length of track to be dealt with. Then, by use of mild-steel hook-bars, hooked into the aforementioned eye-bolts in the roof, and chain-blocks and slings, the length of sleepers and joists was lifted clear of the flat wagon and left suspended from the roof. The flat wagon was then moved clear and the existing length of track below dismantled. Enough ballast was removed to make room for the joists under the sleepers and for the timber sill-pieces, to be inserted at points where trestles would come, after which the suspended length was lowered into position. The old running- and current-rails were replaced

and the track made good for traffic next morning. The old sleepers and chairs were then loaded into the works-train for use on the next length to be dealt with.

Having inserted all the rolled steel joists under the sleepers, the removal of the old invert was taken in hand. At first a narrow heading was driven across under the old invert from the partly constructed invert on each side, and, just alongside a trestle-position, a chase cut through the old invert. Then the service-joist arrangement, shown in Figs. 14 and Figs. 15, Plate 2, was brought into use. A long joist was inserted in the chase and under the longitudinal 8-inch by 6-inch rolled steel joists, the outer ends being supported from the invert already completed at the sides. This joist was supported in the centre of the tracks, by means of a stirrup with adjustable hanging bolts, from a joist running longitudinally overhead between the tracks, which was of sufficient length to span two trestle bays.

After one trestle was erected, the rear end of this joist was supported off this trestle head-tree and the front end supported, well forward on the old invert, beyond two trestle-positions ahead. When these service joists were positioned, and taking their load, the invert was broken out just short of the next sill, the ground excavated, a length of new invert concreted and a completed trestle erected, enough sand being inserted in the foot-blocks to ensure the trestle being tight up to the tracks and doing its job as the service joists were moved ahead. The next night the centre joist was moved forward to its next position and the transverse joists moved up to behind the next sill; the next 8-foot length of old invert was cut away and the new invert put in. During the next night, after traffic-hours, the cross joists were moved just beyond the next trestle-position and then the next trestle was erected, and so on, until the whole of the new invert was completed and the whole length of tracks on trestles (Figs. 15, Plate 2).

The New Station Structures.

As soon as this work was far enough ahead, work was started on the new platforms, staircases, side portions of the booking halls, and entrances at road-levels, the subway connexion for which having already been dealt with. Also, staff accommodation and finishings were put in hand, ready for the new station to be opened at Changeover I. At the old station-site at the same time as the new invert and track-trestles were put in, the temporary timber platforms were also placed on similar but lighter trestles.

The New South Curve.

Meanwhile the work on the new south curve covered way and at the Minories junction (Fig. 2, Plate 1) had also been sufficiently com-

pleted ready for the opening for traffic at Changeover II when the final layout was to be brought into operation.

A small hoarding in the centre of the street near Mansell street on the site of the old City Corporation's conveniences was the only road-occupation permitted in Aldgate High street.

The South Curve: North Wall.

After opening temporary public conveniences, built on a part of the newly girdered area at the south side of the road over the Minories junction, the old lavatories were stripped and a wide heading driven under the road-surface, on the line of the north side wall of the new covered way. When the heading was sufficiently completed, side-headings were broached out at intervals to find the back of the old tunnel, and, sinking at these points from the bottom of the headings, a piece of the new wall was constructed in one with a mass-concrete buttress against the old tunnel-wall, to take any thrust from the old tunnel-arch. Intervening spaces were then excavated and concreted and the whole of the north wall completed.

It will be noticed on *Fig. 16* (p. 526) that the new north wall was positioned here on the site of the existing City Corporation's Aldgate sewer, which had to be diverted to a new position between the old and new tunnels, and which somewhat complicated the procedure.

It will also be observed that the old tunnel-wall had to be underpinned before constructing the new sewer; this was done in short lengths in the usual manner.

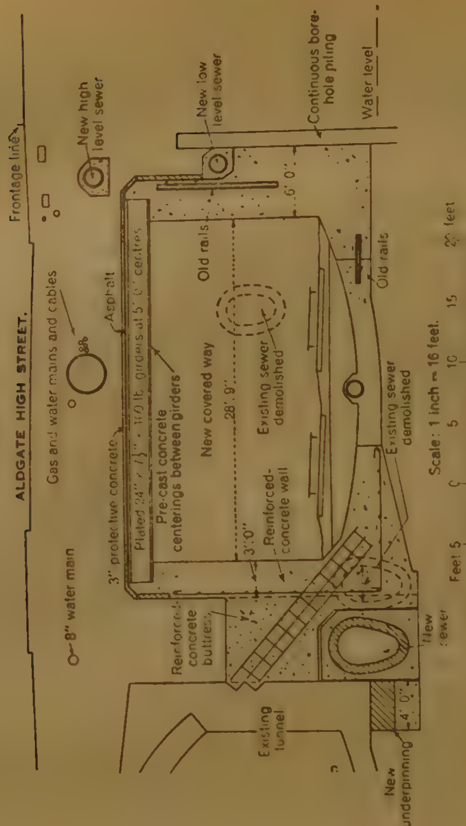
The South Curve: South Wall.

The work on the south wall of the new south-curve covered way was started at the same time as the north wall and on similar lines, but here it was necessary to take extra care of the properties on the south side of Aldgate High street as the work proceeded. The eastern half of this wall is close in front of the foundations of the buildings, and, except for taking part of the cellars of one building, it was possible to avoid interference with the buildings themselves. This was achieved by first excavating along the front of the foundations, down to basement-levels only; the top of the heading in this case being temporary timber decking laid over the pavement- and forecourt-areas affected.

From this level down to some feet below the level of the bottom of the new retaining wall, a continuous row of concrete bore-hole piles was inserted along the front of the old building-foundations, thus avoiding the risky business of underpinning the old unsatisfactory foundations of the buildings in bad ground.

As soon as the piles were in, the wall excavation proceeded by sinking from the bottom of the heading in front of the piles a series of pits, casting

Fig. 16.



New South Curve (Section HH, Figs. 2, Plate 1).

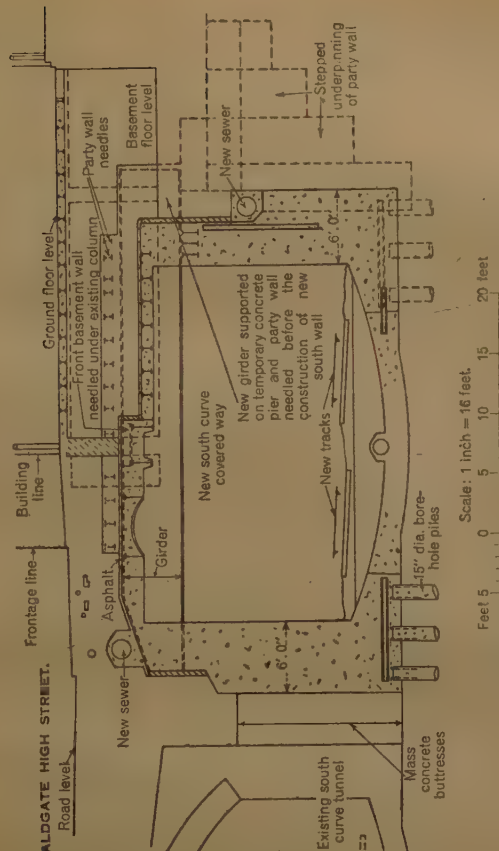
a length of concrete wall in each of these pits, and then sinking for the intervening lengths of wall.

South Curve: Underpinning Property.

The western end of this new south wall is situated wholly under buildings, and here, as seen from the cross section (*Fig. 17*), a rather different procedure and design was adopted. Trial-holes had shown that the foundations of the party-walls of these old buildings were not very satisfactory and on poor ground. Also signs of previous settlement were abundant, the brickwork being much cracked and loose, and very much out of plumb.

It was considered possible to transfer these buildings safely on to the new railway structure, only by avoidance of all intermediate temporary stages of supporting the side and party-walls, and, for that reason, th

Fig. 17.



New South Curve (Section JJ, Figs. 2, Plate 1).

main steel girders of the tunnel-roof were positioned to come immediately alongside the party-walls, and were kept high enough in the basements to be wholly above the old wall-foundations.

These girders being designed to take not only the existing building-loads, but also to take such future high buildings as modern practice would require (if the site were developed), this additional stiffness was taken advantage of. The girders were made with extra length at their southern ends, extending beyond their proper bearings, on the south wall of the tunnel, to temporary bearings on the underpinning piers referred to later. This method was adopted to transfer the weight of the buildings to the new permanent girders before making any extensive excavations under the party-walls for the new retaining wall and sewer.

Before actually placing these girders into position, the new north wall had been completed in the road-headings, and, also, adjacent to the back of the new south wall, a pit 4 feet wide had been taken out under

each old party-wall, and extending 4 feet on either side of the foundation of the latter. This pit was taken down to below the final wall-foundation level and filled with concrete, the party-wall itself being pinned up in new brickwork off this concrete.

As will be seen from *Fig. 17*, the party-walls were also further underpinned behind this, the underpinning stepping up at 45 degrees. Bearings were then prepared on the first block of concrete for the projecting ends of the main girders. For the erection of these girders, nights were chosen when the butchers' markets were closed, and by taking out part of the ground-floor of the shops, and breaking out a heading in front of the shops to meet the north wall heading, the girders were drawn into their positions at the side of the party-walls, one end bearing on the north retaining wall and on the underpinning piers at the other. Then the short steel-joist cross needles, seen in *Fig. 17*, were inserted into the brickwork and carefully wedged up to carry the party-walls.

When several of the party-walls had thus been dealt with, pits for sections of the south walls proper were excavated and the walls cast, certain reinforced-concrete borehole-piles being inserted at the bottom of the walls to allow for any future building-loads already referred to.

The girders were then wedged tight and grouted on their proper bearings on the south wall. As the girders were erected, the road surface above their north ends was packed up from them and the intervening portions of dumpling taken out to just below new tunnel-roof level, the road surface being propped off the ground temporarily. The intervening panels of filler-joists were inserted and the roof completed, the road surface being finally supported off the new roof by brick piers at intervals with weak concrete filling in the intervening spaces, certain provisions being made for pipe- and mains-diversions as the work proceeded.

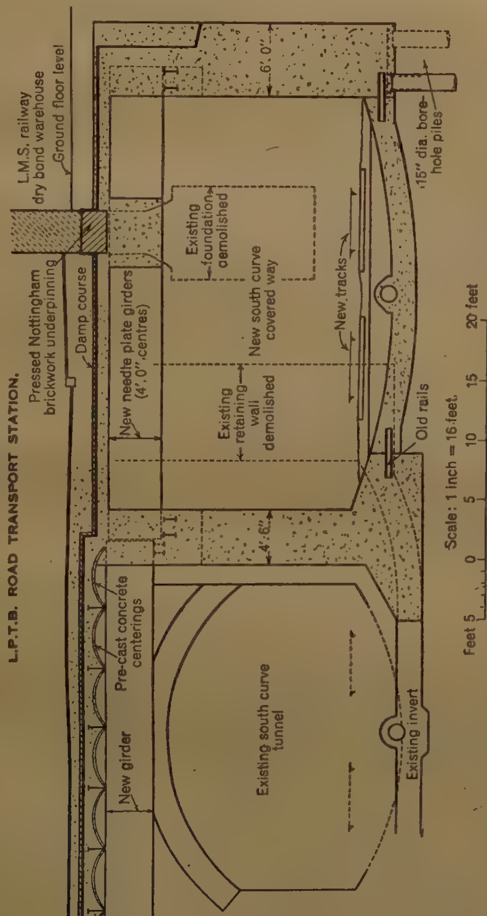
The new sewers were then built, and the house-connexions, which had temporarily been carried through the new wall to the old sewer in the tunnel dumpling were transferred to the new sewer and the wall made good.

Within the buildings, the roof of the new tunnel was constructed of filler-joists and concrete resting on the lower flanges of the main girders, thus providing a basement-floor for each of the buildings. Having completed the new tunnel-roof, the removal of the dumpling excavation was commenced. A mechanical excavator was used, working from the west end. The excavation was taken right down to the invert formation-level, the new concrete invert closely following the excavator. As this excavation proceeded, the disused sewer and the old building-foundations within the new tunnel were demolished.

At the extreme west end of this work, the corner of a high and heavy brick warehouse building (a portion of the London Midland and Scottish Railway's Haydons Square depot) projected over half the roof of this new covered-way (Fig. 2, Plate 1, and *Fig. 18*).

This building was of massive construction and imposed a load of about 1,000 tons on the new tunnel-wall. Here the roof-girders of the new covered way, which were at 4-foot centres, were introduced as "needles," through the foundations of the building, and the holes then made good round the girders in concrete and brickwork. As each girder was intro-

Fig. 18.



NEW SOUTH CURVE AT MINORIES JUNCTION (SECTION KK, FIGS. 2, PLATE 1).

duced, it was wedged up from its bearing on the wall to a predetermined dead-load deflexion. The wall supporting the ends of girders was cast in one piece 75 feet in length to distribute the load longitudinally, a trench of this length having been opened up by breaking through the existing foundations on each side of the building.

Twenty-nine reinforced-concrete bore-hole-type piles were placed below the wall to carry the heavy load, and steel reinforcement was introduced

in the bottom of the wall to ensure that the wall acted as a bea transmitting the load evenly to all piles. A bearing-test was carried ou on one of these piles, where it came directly under the wall-foundation of the old building. A hydraulic jack was used and the pile was twi tested up to 50 tons load (working load 35 tons). It was found that 50 tons the actual movement at the top of the pile was $\frac{1}{32}$ inch, and removal of the load, the residual set was only $\frac{1}{100}$ inch.

The Road-Transport Station.

Reference to the same cross section, *Fig. 18*, gives an illustration of th type of steel-and-concrete deck that was adopted for covering the larg area over the Minories railway junction for the new Road Transpo station. Plate-girders were supported over the railway between th retaining walls at such points where steel stanchions and foundations coul be placed to avoid both the old and the new track-layouts. On the main girders the deck itself was constructed of deep steel joists space 5 feet apart, supporting the standard pre-cast-concrete jack-arch centre before described. On these centres the concrete deck was cast, and th whole covered with a bituminized felt damp-course and blue-brick pavior upon which the road surfacing was laid.

Changeover I.

This changeover, already referred to, was effected between the runnin of the last train on Saturday night, at 1 a.m. Sunday morning, and th first train on the following Monday morning at 5 a.m. The Sunday train were not run between Aldgate (Circle) station and Whitechapel station, special omnibus and tramway service being introduced to take railway passengers between these two stations.

This changeover brought the new station into use and closed the ol station, but maintained the track-layout through the old station-site, th old south-curve tunnel, and at the Minories junction.

It entailed lowering the tracks throughout the works (some 1,400 feet o double track) to their final levels, completing the centre portion of th west ticket hall, and demolition of the old platforms and stairways Though the track-trestles and the temporary timber-platforms had bee designed with a special view to their quick removal at this changeover, i view of the tremendous amount of work involved in the short tim available, it was thought desirable to stage a small rehearsal on on Saturday night, a month beforehand, to discover any unforeseen difficulties A single 60-foot length of double track was selected and each of th London Passenger Transport Board's Departments concerned, and bot Contractors, carried out a complete sequence of the operations they wer expected to perform on the actual changeover. The squads consiste

mainly of men who would be in charge of gangs, and very valuable and reassuring experience was thus gained by all concerned.

For the purpose of the changeover itself, the tracks were split up into five sections, each consisting of a 240-foot length of double track and an odd length at the extreme east end, where the new gradient joins the old track-levels, which was dealt with by the Permanent Way Department separately. The portion situated west of the new station through the old station-site, consisting of two of these track sections, was dealt with by thirty-two sets of special lifting tackle, and the portion for the new station-site and eastwards, consisting of three track-sections, by another thirty-two sets of similar tackle.

It will be seen by referring to Figs. 14, Plate 2, that the tackle consisted of $1\frac{1}{8}$-inch diameter steel hook-bars hooked into the large eyebolts which had been left in the new roof, and on each was suspended a 2-ton chain-block and a short chain-sling, only long enough to slip under the running rails and back to the hook of the block. The lengths of the hook-bars were made such that the chain-blocks could be hooked on easily from an ordinary platelayer's truck running on the track.

During the day before the changeover, the lifting tackles were all placed in position alongside the points where they would be first erected, and all other plant and apparatus, which would be required, was similarly positioned. At the commencement of the changeover, immediately the last train had gone and current had been taken off the tracks, permanent-way gangs placed one of the trucks on each track of the first two 240-foot sections to be dealt with. As these trucks were pushed along the tracks, men at the sides handed the lifting tackles to men on the trucks who hung them from the eyebolts in the roof.

At the same time other men cut the bonds on the current- and running-rails, at the ends of the sections, and unfished the rails at these points. Further squads of men of the London Passenger Transport Board's Signal and other Departments disconnected all signal-wires, air-pipes, and other track-equipment. Squads of contractors' men simultaneously took out the bolts connecting the track-sleepers to the temporary longitudinal 8-inch by 6-inch rolled steel joists, and also took off the bracings of the trestles. For undoing the bolts, free use of pneumatic tools was made with considerable saving of time, the tools being used on the bolt-head, and the nut being held with a spanner. The bolts were thus at the same time also loosened in the timber, and no time was lost in drawing stubborn bolts. While the bolts were being taken out, the permanent-way gangs lifted, by means of the lifting tackle, the whole length of each section of outer current-rail into the centre of the tracks, and then slung each running-rail, ready for lifting the section. As soon as all sleeper-bolts were out, the men at all lifting blocks, working in concert, lifted the 240-foot section of double tracks, complete with sleepers, etc., about 1 foot clear of the trestles.

The longitudinal joists were then slid out from under the tracks and the trestles dismantled, the material being taken up by cranes stationed in the road, through winding bays left in the south side of the new tunnel roof, and loaded into lorries or stacked for removal later.

When the trestles were clear, the tracks were lowered on to the ballast bed which was roughly spread beforehand to the level of the underside of sleepers. The outer current-rails were then lifted back on to the insulator-blocks.

Before raising the first lengths of tracks the platelayers' trucks used for erecting the lifting tackle had been pushed on to the next section. As soon as the first section was down, more trucks were placed on this section with a specially made ladder-platform on each to bring them level with those on the higher tracks. From these, men unhooked the lifting tackle over the lowered section, and transferred the tackle to men on the higher section to erect in the next lifting positions. Thus the transfer of the sixty-four sets of tackle, from the first two sections, to the subsequent sections was effected with no loss of time. Owing to the varying heights of the roofs, different lengths of hook-bars were supplied and each type was coloured differently in accordance with a key plan, so that one man on each track could direct the particular bar to its correct position. As soon as the two adjacent sections of track were down, rails were connected up, current-bonds replaced, the new signal-apparatus, etc., which had previously been partly prepared, was placed in position, ballast-trains were run over the sections to complete track-ballasting, and fettling up the track was commenced. While the tracks were being lowered, other squads of men were dismantling the temporary timber platforms, staircases, etc., at the old station, the material being cleared away in a similar manner. Meanwhile, at the new station, the centre portion of the new west ticket hall was being dealt with. The floor steelwork-members had all been laid out beforehand and were quickly erected, after which the centre portion was temporarily decked with timber. The ticket booth was moved into position and control-barriers, ticket machines, lighting, etc., were also assembled.

In preparing for this changeover, carefully programmed instructions were drawn up for every one concerned. Limiting times, based on the rehearsal, were given for each operation, and certain margins of time allowed ; also, in staffing the gangs on duty at various periods during the weekend, a reserve was arranged to be available, if required, by overlapping the shifts. However, the whole of the operations from start to finish went like clockwork, every man knowing exactly what was expected of him, and it was not necessary to draw on reserves at any time.

The tracks were scheduled to be all down by midnight on Sunday and they were, in fact, all down by 6 p.m. that evening. Thus ample time was available to make good the tracks, test the new signalling, complete the new station-equipment, and clear away all the old material, so that there was

no hitch in the traffic-operations on Monday morning. This, considering the enormous quantities of material involved, was very satisfactory, and although there were nearly 900 men employed on the job, no confusion occurred.

Changeover II.

The second stage of the works only lasted 4 weeks, between changeover I and changeover II. During this time, the new tracks, with signalling equipment, etc., were laid in the new south-curve covered way. The new track-connexions just west of the new station, and at the Minories junction, were inserted, and the tracks slewed in the old station-site to the final positions, and, on one Saturday night, between 1 a.m. and 5 a.m. on the Sunday morning, the changeover to the new final layout was effected.

Acknowledgements.

The Author, as New Works Engineer of the London Passenger Transport Board, was responsible for the design and execution of these works under the Chief Engineer of the Board.

The satisfactory carrying out of this important work reflects great credit on all those concerned, and the Author wishes to make acknowledgement for the help, co-operation, and ingenuity displayed, and also in respect of certain data published in this Paper, to the following :—

Messrs. John Cochrane and Sons, Ltd. (Contractors for the new station and works immediately east and west thereof).

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He also wishes to acknowledge the help of the Chief Engineers and staffs of the following Local Authorities and Service Undertakers :—

London County Council.
Corporation of the City of London.
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General Post Office.
Metropolitan Water Board.
Gas, Light and Coke Company.
Commercial Gas Company.
London Power Company.
Central London Electric Company.
City of London Electric Lighting Company.
London Hydraulic Power Company.

In addition, his thanks were due to the staffs of the London Passenger Transport Board Operating, Architects, Permanent Way, Signals, and Power Departments, and to the Resident Engineer, Mr. J. W. Carswell, who was so ably assisted by Mr. A. Sanderson, Mr. T. K. Sargent, and Mr. H. G. Follenfant, B.Sc. (Eng.), Assoc. M. Inst. C.E., as assistants.

The Paper is accompanied by seventeen sheets of drawings and three photographs, from some of which Plates 1 and 2, the Figures in the Text, and the half-tone page-plate have been prepared.

Discussion.

The Author showed a number of lantern-slides illustrating the works described in his Paper, and gave a sound-impression, by means of a gramophone-record, of the preparation for the first changeover.

Mr. Raymond Carpmael remarked that the Author, in indicating the necessity for the work described, said that only a bold scheme could solve the problems. There would be general agreement that a bold scheme had been evolved. Having indicated the problems, the Author proceeded to set out the obstacles, each of which presented many problems requiring the closest consideration. The principal concern of railway engineers when carrying out alterations to permanent way and stations was as a rule more or less confined to the maintenance of railway services, but the works described by the Author involved the maintenance of the continuity of the following services: the London Passenger Transport Board's own railway (with very brief occupations, generally of about 5 hours), the street-traffic overhead, including conduit-system trams, gas, water, drainage, electric power and light, and telegraphs and telephones, all in a very congested space. The work was also complicated, as had been stated, by the limitations imposed on interference with the business of traders. In fact, as the Author said, the methods of carrying out the work to overcome those difficulties really dominated the design; that had been brought out very clearly in the Paper.

Mr. Carpmael was very impressed by the models which had been made. They enabled the whole position to be visualized to an extent which could not have been possible by any other means. He could not improve on what the Author said about them in the section of the Paper headed "The Models" (p. 507). It would be of interest if the Author would say how many of the pipes had been located by the digging of exploratory trenches and how many had not been located at all until the work was actually in progress. It would appear that much information with regard to them was available, and in that respect the work differed from earlier works; the engineers responsible for the construction of the Central London Railway would envy the Author in that he could settle the progress-programmes for the works carried out with a certain degree of precision.

No details were given in the Paper of the plant employed in carrying out the excavations, which, in the circumstances described, was bound to have involved some difficulties. If any special types of plant peculiar to the job had been employed, details of them would be of interest.

The method adopted to avoid settlement-cracks of the main concrete walls was of interest, as being that adopted by the late Sir James Inglis,

Past-President Inst. C.E., in the construction of the parapet-wall of th breakwater of Fishguard harbour, although in that case the wall wa built on a rubble base and the object was simply to allow the free settlemen of each section independent of its neighbours. The same dimension, 2 feet, was selected, and the special V-jointed ends were thickly coated wit putty mortar before adjoining lengths were cast.

The Author referred to the use of pre-cast concrete jack-arch centering to avoid suspended timber centres. Mr. Carpmael had for some years use that type of jack arch, and had standardized it to suit various widths. I railway maintenance their use often simplified the work. It would b of interest to hear whether or not the Author had had any trouble wit distortion or cracking of any of the girders due to the very unusual metho of handling them.

Among the many difficult operations carried out, that of the lowerin of some 1,400 feet of track was perhaps as difficult as any, in that it had t be carried out in a very short time. It was axiomatic in good railwa engineering practice that the complete success of work of the character i question could be assured only by the efficiency of the preparatory arrange ments, and the Author had emphasized that in presenting his Paper.

Mr. Arthur R. Cooper said that, having been Chief Engineer durin about two-thirds of the work, there were a few matters which he would lik to refer to.

In the initial stages, arrangements had been made for the engineer who were going to be actively engaged on the work on the site to take part in the design, in the preparation of the drawings and specificatio and in particular in getting out the various stages of the work. That gav them an intimate knowledge of what they had to construct and of th reasons why certain methods were being adopted. With an intricate work such as that in question, with unforeseen difficulties liable to arise at an time, great benefit had been found from adopting that policy and from bringing the men into the work at a very early stage.

The Author had referred to two contracts being let for the work. A the time when tenders were invited for the work it was very difficult to ge steelwork promptly, and precedence was therefore given to finishing off th steelwork drawings at as early a date as possible. The contract for th steelwork was later made a sub-contract to the main contract. As woul be seen from the Paper, the erection of that steelwork was a very com plicated matter, and no little study was given to the method by which i should be carried out. Before that contract was let, all the firms wh were going to be invited to tender for the main contract were seen an their general concurrence obtained with the methods of steel-erectio proposed. He considered that that was a wise precaution, because i guaranteed that there would be no misunderstanding or difference o opinion between the main contractor and his steelwork sub-contractor.

Reference was also made in the Paper to various schemes having bee

prepared. As early as 1913 powers had been obtained for the lengthening of the platforms at the old station. They were originally built for six-car trains only, and eight-car trains were running. The only precaution which it was possible to take at that time was to advise that only six cars could be used for passengers getting out or joining the train at that station, and horizontal ramps were put in in the tunnels at the end of the platforms, so that any one trying to get out would be protected. In 1920, other schemes were prepared on the basis of lengthening the platforms, but none of them embodied the bold policy of constructing a new station at a good traffic-centre and of doing away with two old stations which were not very well placed. The scheme which was eventually carried out also had the great advantage that it increased the factor of safety of traffic-working, as originally there were certain signal overlaps and arrangements which had not the factor of safety which was desired.

It was too early yet to give any particulars with regard to the effect on the traffic of doing away with the two old stations and building the new one. Naturally, during the construction-work the traffic fell at the two old stations, but he had been informed that during the last month the traffic at the new station had built up to the total of the two adjoining stations which had been closed down.

Reference had been made to the models. The making of models had always been a practice of the Underground railways. In the early days of the tube railways, Lord Ashfield had suggested that full-size models should be made of intricate passages and tube-work, and several such models were built and housed in a big hall which the railways owned at Earl's Court. That enabled the traffic officers to wander about the passages in those full-size models and to judge the actual requirements. That was of especial benefit so far as tube-work was concerned, because the difficulty and cost of making alterations in tube-construction after it had been completed would be appreciated.

The change-over had been rightly referred to by Mr. Carpmael as having been very carefully planned and very successfully carried out. It had been carried out under Mr. Cooper's successor, Mr. V. A. M. Robertson, M. Inst. C.E., who took a keen interest, with the Author, in working it out and making a great success of it. It might be mentioned that the whole of the intricate work which was done, and which covered a period of 2 years, was carried out without any mishap at all. That was a very great tribute to the engineers, the contractors, and all those associated with the work.

There was one incident which Mr. Cooper would recall. There were large gas-mains under the street which were being bagged off as a means of closing them, and unfortunately one of them caught alight. The fire brigade was called, but it was considered too risky to put the fire out; there were tunnels and cavities alongside where the gas might collect. It took 15 hours to bag off that supply, because four or five large mains had to be bagged off before the one concerned could be dealt with, and

during that time the fire-brigade continued to play water on the surrounding timber and excavation.

Mr. H. Alker Tripp observed that, as the Assistant Commissioner of Police, he was in charge of traffic-circulation in London. The Author had spoken lightly of street-occupation, but, in the case under discussion, traffic would have stopped unless drastic steps had been taken. Some of the traffic had to be accommodated elsewhere, and it might be of interest to refer to the arrangements made. During the time that the southern half of Whitechapel High street was occupied by hoardings and works, the traffic had been brought into a uni-directional flow by the Whitechapel High street westbound traffic being brought down Church lane (Fig. 2, Plate 1), turned right along Alie street, then right into Mansell street, and finally left into Aldgate High street. The eastbound traffic used the north side of Whitechapel High street. Pedestrians, being unsighted by the hoardings, were exposed to special dangers; barriers were therefore erected alongside the pavements and the tram-lines to prevent the promiscuous crossing of pedestrians, and gates in those barriers were provided at five points. At those points London Passenger Transport Board men were stationed in control of signal-lights, which they operated as the police stopped the traffic, thus enabling pedestrians to cross. Chains were drawn across to prevent pedestrians crossing when the lights were against them. That system came into operation when the hoardings were on the south side of Whitechapel High street; it was not in operation when the hoardings were on the north side. There was a 60-per-cent. reduction in pedestrian casualties when those arrangements were in operation.

Mr. C. M. Norrie observed that when the works had been started, a very eminent engineer had said to him that works of that description should be carried out without interfering with the road surface at all. Mr. Norrie had been concerned with the work for 2 years, however, and his view was that if it had been possible to carry out such a work without disturbing the road surface, it would have been at enormous cost and would have involved a very great extension of the time taken for carrying it out. When his firm had tendered for the works, they had naturally found it very difficult to estimate the costs and to price their tender, and he would like to thank the railway company, and the railway engineers more especially, for allowing them to look at the wonderful models which were made before they tendered.

Another matter to which he would like to refer was the way in which the quantities were taken out. An engineer had generally to deal only with his own quantities, but contractors had to deal with the quantities of a variety of engineers, and that presented some difficulty at times. He would like to add—although that might not relate to the work described in the Paper—that there had been a growing tendency, no doubt on the part of employers, to insist on the quantities being got out by independent quantity surveyors, and that often gave the contractor a great deal of

additional work. The quantity surveyor was seldom able to visualize a heavy constructional work in the same way as an engineer, and the quantity surveyor's method of itemizing in too great or too little detail was often the cause of a great deal of extra work to contractors who were tendering; Mr. Norrie therefore considered that the engineer who had designed the works, and who knew how they were going to be carried out, should prepare the bill of quantities.

On behalf of his firm, he would like to mention and congratulate their agent, Mr. A. B. Gladwell, and his very able assistant, Mr. G. Ford, on their untiring efforts during the construction of the works, and to thank Mr. J. W. Carswell, the Resident Engineer, for his help.

Mr. T. H. Seaton had been concerned in the reconstruction under traffic of a number of stations, and agreed with Mr. Carpmael and the Author on the importance of making the most complete preliminary investigations, and also of preparing a complete programme in detail of works such as those described in the Paper. The models used were very instructive, as no drawings could show so clearly the works affected, and in particular public services such as sewers and water- and gas-mains and electric cables. In works of the character in question it could not be too strongly emphasized how important it was that the fullest information should be obtained of those public services before the works were actually designed, as otherwise it might be found necessary to redesign parts of them at a critical stage of their construction.

In carrying out railway works, the co-operation of the departments concerned was vital, and might be secured by frequent meetings of representatives at which the various stages of the work were discussed. From those meetings, a programme and stage plans could be prepared and circulated to all departments. That procedure had evidently been followed in the works described in the Paper, and had played no small part in ensuring the success which had been achieved.

A feature of note was the manner in which the sympathy of the public had been enlisted in connexion with any inconvenience to which they might be put. That had been achieved by pictorial posters describing the work which was being done by the engineers. Engineering works which affected the working of a railway were bound at times to cause delays, and the London & North Eastern Railway had found that, by taking the public into their confidence and explaining what was being done, complaints had been materially reduced.

A particularly valuable part of the Paper was the description and drawings of the temporary works, the character and extent of which were unique. Much consideration had evidently been given to their design, especially with regard to the rapidity of dismantling at the changeover periods, and the rehearsal carried out was an undoubted safeguard against any points in connexion with the work being overlooked. Apart from the description of the permanent works, the Paper was undoubtedly of par-

ticular value for the description of the changeover operations, and was to be recommended for careful perusal by railway engineers who might in the future have to carry out works of the character in question.

Mr. H. G. Follenfant remarked that it would be readily appreciated that from beginning to end a feature of the works was carefully planned organization. There were few operations which were unaffected by such considerations as the maintenance of road- and rail-traffic, pipes, mains and sewers. Even such events as Christmas shopping and the illuminations at Southend had their effect upon the programme. The need for avoiding interference with local trade made some parts of the works very difficult. For example, special methods were necessary when shopkeepers had such keen business instinct that they sold meat during the night and such things as portmanteaux during the day in the same shop.

Those who had some share in the preparation of the contracts endeavoured to do all that was possible to present a picture of the works to the contractors, and to set out in narrative form a schedule of how it was visualized that the works would be carried out, although in some cases that had of necessity to be in outline. In the second contract, for instance, the condition and position of the foundations of some of the buildings were unknown until the ground was opened up, and the detailed design had to be done as the work progressed. A small drawing office was maintained in the Resident Engineer's office for that purpose.

It was, however, to the main contractors that the greatest credit was due for the way in which they organized in detail and carried through successfully the very complicated series of operations. The climax of their work was the big changeover from the old to the new station, and he would like to add a word or two to the description given in the Paper.

The broad outline of the scheme had been decided and set out in the specification. The date had to be fixed to suit the Traffic Department: not too near the summer holiday season and not too near Christmas. Progress was frequently reviewed in the earlier stages of the works, and the actual date had to be chosen in relation not only to the works at Aldgate but to changeovers on other lines where large gangs of the Board's staff were employed. It was difficult to decide the length of time to be allowed: as the time drew near, and especially after the rehearsal, everyone felt confident that from Saturday night to Monday morning was ample time, provided that there were no hitches, but that it would be unwise to make the widely advertised traffic-arrangements for a shorter occupation. The programme was accordingly drawn up on the basis of the maximum time that could be allowed for the demolition and removal of the trestles and the lowering of the tracks, allowing a minimum period of from midnight to 4.45 a.m. for completing the signalling and fettling of tracks.

After much consideration between the railway departments concerned and the contractors, a programme was agreed. Various methods of depicting the programme were considered, but the method found most

useful in the end was to set out on ordinary foolscap a series of typewritten statements covering every operation in chronological order. Opposite the statements in two columns were the limiting time to be allowed and the name of the assistant responsible for supervising the operation. All those in a supervisory capacity—foremen, inspectors, and charge-hands—had copies and were responsible for seeing that each individual man knew his job. The result was even better than had been hoped, and there was a noticeable atmosphere of enthusiasm, as if each man felt that upon him individually depended the success of the undertaking.

Mr. H. W. S. Husbands said that, as the Author had pointed out, none of the early schemes complied with all the requirements, and only a bold scheme sufficed to get rid of all the difficulties and to give the full advantage from the expense undertaken. The Board were also interested in traffic on the roads, and it seemed a pity that such a scheme could not have been combined with the provision of a fly-over junction for road traffic, which would have done away with congestion at that point. There was no doubt that traffic-congestion in London would never be eliminated unless fly-overs were provided at the main junctions. The roundabout would not solve the problem. At a level four-way road junction he estimated the lost capacity at three-quarters of the road capacity. There had been a proposal at Aldgate to construct a parallel road, but that would not get rid of the cross-traffic difficulty, and he did not think that much advantage would be obtained by diverting the traffic; in fact, it was impossible to divert traffic from central London. He hoped that when the Board were considering other schemes at important junctions they would at any rate see that when the designs were prepared the station was not sited in such a manner as to make it more difficult or more expensive to construct a fly-over junction at such a spot.

⁂ **Mr. R. H. Cunningham** observed that the gradient through the new station was not clear in Fig. 2, Plate 1; it was shown as level at the east end and 1 in 260 at the west. The new Metropolitan District Railway lines on the site of the old station were shown to be on gradients of 1 in 57·6 and 1 in 52, and probably an intermediate gradient-post had been omitted. With regard to the 1 in 40 gradient at the east end of the new station, the distance between the gradient-posts was 228 feet, giving a fall of 5·7 feet from the higher level to the station, as compared with the figure of 7 feet shown in Fig. 5, Plate 1.

He would like to obtain some information in regard to the vertical curves in the 1-in-40 gradient. In order to insert two equal vertical curves meeting at a point of reverse curvature half-way down the gradient, the radius of each curve would have to be about 9,000 feet, but probably in view of the proximity of the gradient to the platforms and the consequent

⁂ This and the succeeding contribution were submitted in writing.—Sec. Inst. C.E.

low speeds the radius of each curve was considerably less. Was th gradient an arbitrary figure, or was it based on conditions of speed?

Mr. H. G. Lloyd observed that in Figs. 7, Plate 1, was shown a heigh of some 16 feet of sandy ballast above invert-level, containing subso water to a depth of about 7 feet. It would be of interest to know wha proportion of the 500 gallons per hour pumped from the sump shown Fig. 5, Plate 1, was subsoil water; experience had shown him that i general there was likely to be an increase of seepage, and some settl ment, where the finest grains of sand were drawn away from the balla by the flow of subsoil water.

The Author, in reply, observed that some 20 years before the wor was commenced the London County Council had laid out very complicate tramway junctions at the intersection of Commercial street, Commerci road, and Whitechapel High street, and records were available of th positions of the shallow pipes, mains, etc., that were then uncovered. Th depths of the larger and deeper mains and sewers were obtained from th service companies, or local authorities, to whom they belonged. Larg sewers, say over 3 feet in diameter, were actually re-surveyed through th sewers from manhole to manhole. Where vital, the positions of main were ascertained beforehand by trial-holes or headings. When the wor was carried out some of the deeper mains were found some distance from the positions shown on the model and drawings. All the service under takings and local authorities did their best to give accurate information and on the whole the information was remarkably correct.

A large percentage of the excavating was carried out by the ol method of hand-filling skips, which were then lifted by crane Pneumatic spades were used to some extent in clay. A smal short-jib mechanical excavator was used at the Minories junction and afterwards for removing part of the dumpling in the new south curve In the work under the shops in Aldgate and Whitchapel High streets 2-foot gauge track was laid to hoisting bays.

Reference was made to possible distortion of the steelwork through th method of erection. No trouble, however, had been experienced from tha cause, as the method of erection was known when each girder was designed and the method of slinging and handling was carefully watched by th assistant responsible. A very close liaison existed between the steelwor sub-contractors and the Resident Engineer's staff, and all methods o erection and handling of steelwork were discussed and agreed beforehand

Mr. Cooper referred to two contracts being let, and also to a steel contract The Author would like to make it clear that there were two main con tractors on the work and a steel contractor appointed by the Board, wh afterwards acted as a sub-contractor to the two main contractors. Th interrelation of the three contractors on the one job was carried throug by the good will and enthusiasm of all concerned.

Mr. Tripp referred to the very useful experiment in pedestrian-contro

hat was carried out. The police authorities were very helpful and their iversion of the eastbound traffic made the works possible; the Board vere glad to have been able to help them in return with their experiment. 'he experiment was being carried a step farther on the large works at resent being undertaken at King's Cross, but members of the public had omplained that the Board had interfered with the right of the public by he installation of some of the pedestrian-controls there. The Author was ure, however, that that pedestrian-control was far the best for all oncerned.

In the Paper he had referred to the advantage of the Resident Engineer aking considerable responsibility in the office for the design before the ontract was let, and he noted that Mr. Norrie strongly approved of the lescription of the works in the specification and the quantities being taken out in the same manner as the drawings were prepared by the engineers who would be responsible for the work. He agreed that the greatest ossible technical information should be available for the contractors when vorks of the nature described were being tendered for. Mr. Norrie also eferred to his agent and staff on the work, and the Author recognized he great help given by the agents and staff of the contractors and the nitiative and ingenuity that they displayed in overcoming the many bstacles on a very restricted site for such large works.

Mr. Seaton referred to the close co-operation between the different lepartments of a railway company when traffic was interfered with. In hat connexion meetings were held weekly (sometimes more often) throughout the duration of the works, when, if necessary, representatives of the lepartments concerned were invited to be present. The Traffic Department had a Liaison Assistant always in touch with the work. Mr. Follenfant gave further useful details of the organization in connexion with the final changeover, and rightly mentioned the enthusiasm of all concerned.

Mr. Husbands complained that the Board had not provided for fly-over junctions for road traffic, but he forgot that that was not the Board's function. He also suggested that all works carried out by the Board should be designed in a manner to facilitate future fly-under junctions; but the Author thought that he had overlooked the fact that at Aldgate East the Board had provided two fly-unders for pedestrian traffic crossing the congested Whitechapel High street, and by lowering their running tracks 7 feet it had made a valuable contribution to any scheme the Local Authorities might bring forward in the future for fly-overs for road traffic.

Mr. Cunningham asked for information about the track-gradients through the new station. Fig. 2, Plate 1, showing the general lay-out, was a reduction from the 20-feet-to-1-inch scale plan, and for clarity certain of the intermediate gradient-posts had been omitted. Those posts were: one immediately west of the east ticket-hall, showing a change from level to 1 in 260; the other was on the two tracks of the south curve between

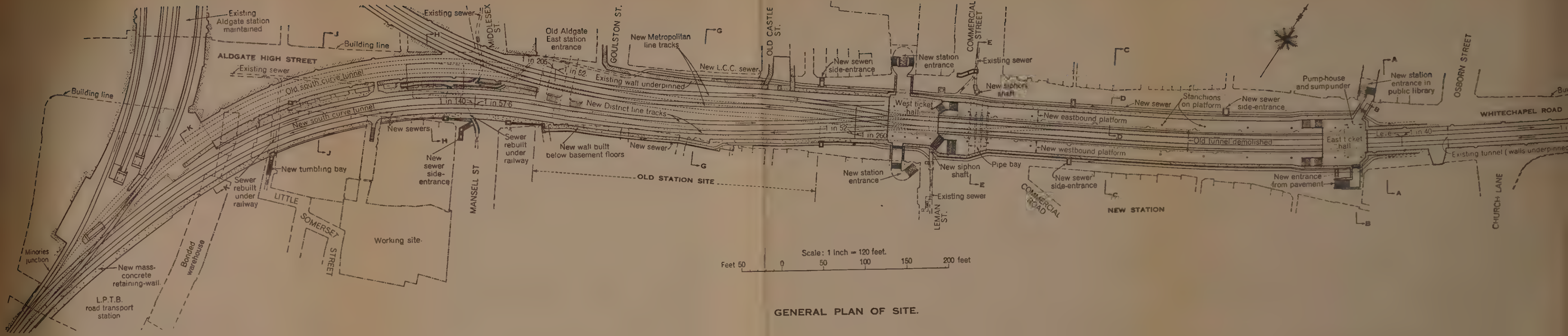
the gradients of 1 in 52 and 1 in 57·6. With those additions the gradient signs were complete. The Board's present practice, wherever possible was to provide a vertical curve at changes of grade of a radius not less tha 40 chains. That curvature had been adopted at Aldgate East.

In reply to Mr. Lloyd, the whole of the water pumped was subso water, but it was by no means wholly from within the area of the new works. The greater proportion was from the track-drains of the old tunnels, both east and west of the new works, which had been connected up to the new drains. The Author suggested that the amount from the new works would be something of the order of 100 gallons per hour. Although the volume mentioned might seem large, the wall- and invert area was also large, and in no case was there more than a very slight trickle through any of the many joints in the concrete invert and lower parts of the walls, and there was nothing that should cause any apprehension regarding robbing the subsoil of its finer particles.

*** The Correspondence on the foregoing Paper will be published in the Institution Journal for October 1939.—Sec. Inst. C.E.

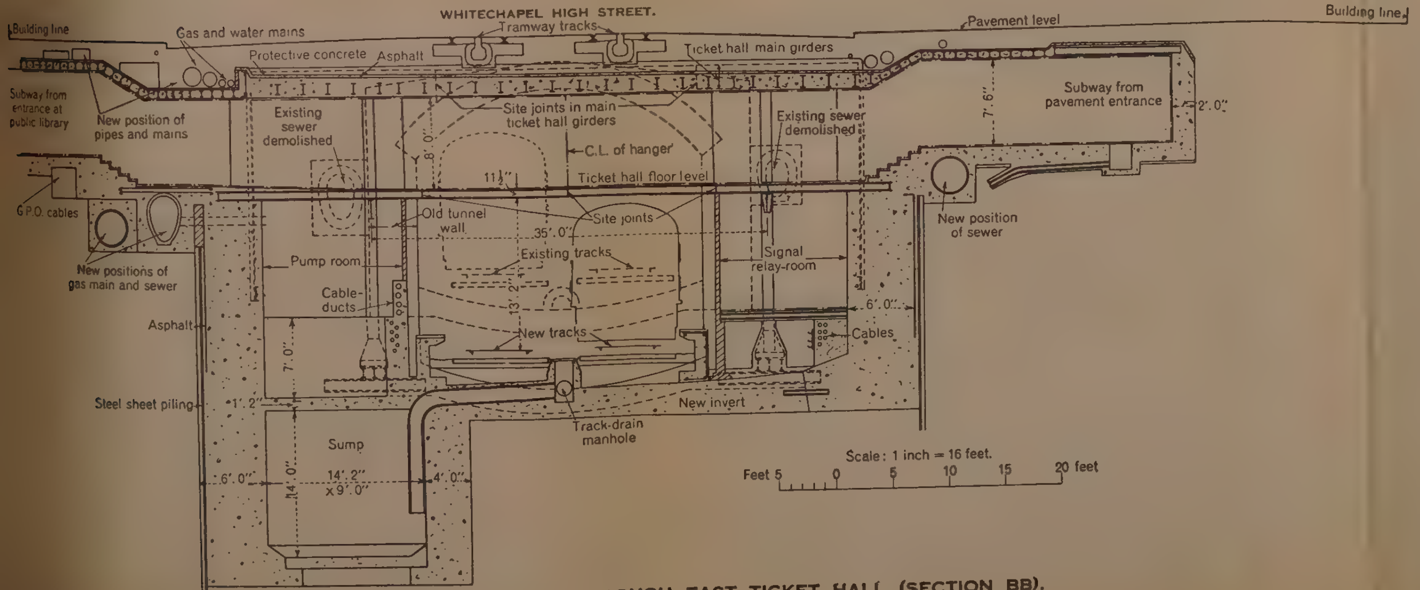
RECONSTRUCTION OF ALDGATE EAST S

Fig: 2.



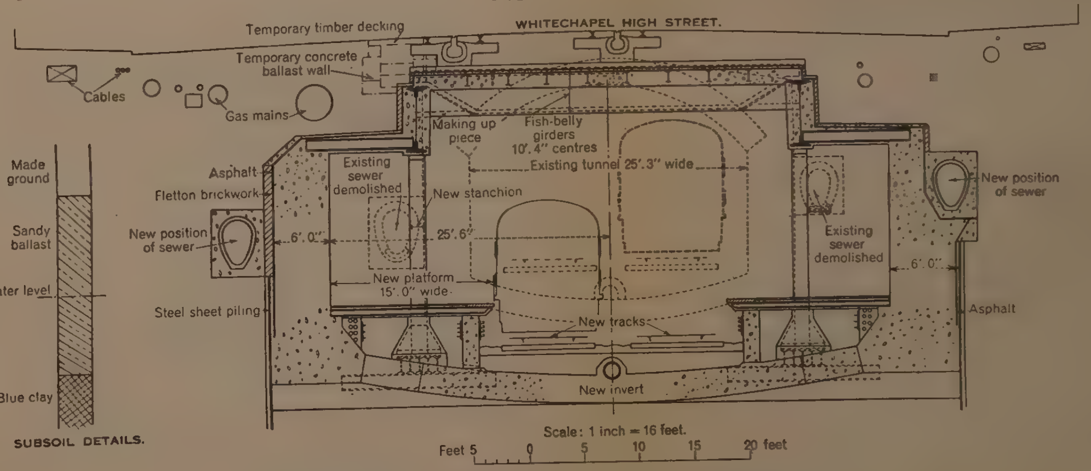
GENERAL PLAN OF SITE.

Fig: 5.



CROSS SECTION THROUGH EAST TICKET HALL (SECTION BB).

Figs: 7.



CROSS SECTION THROUGH NEW STATION (SECTION CC).

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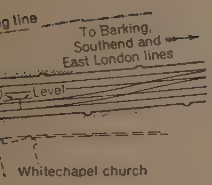
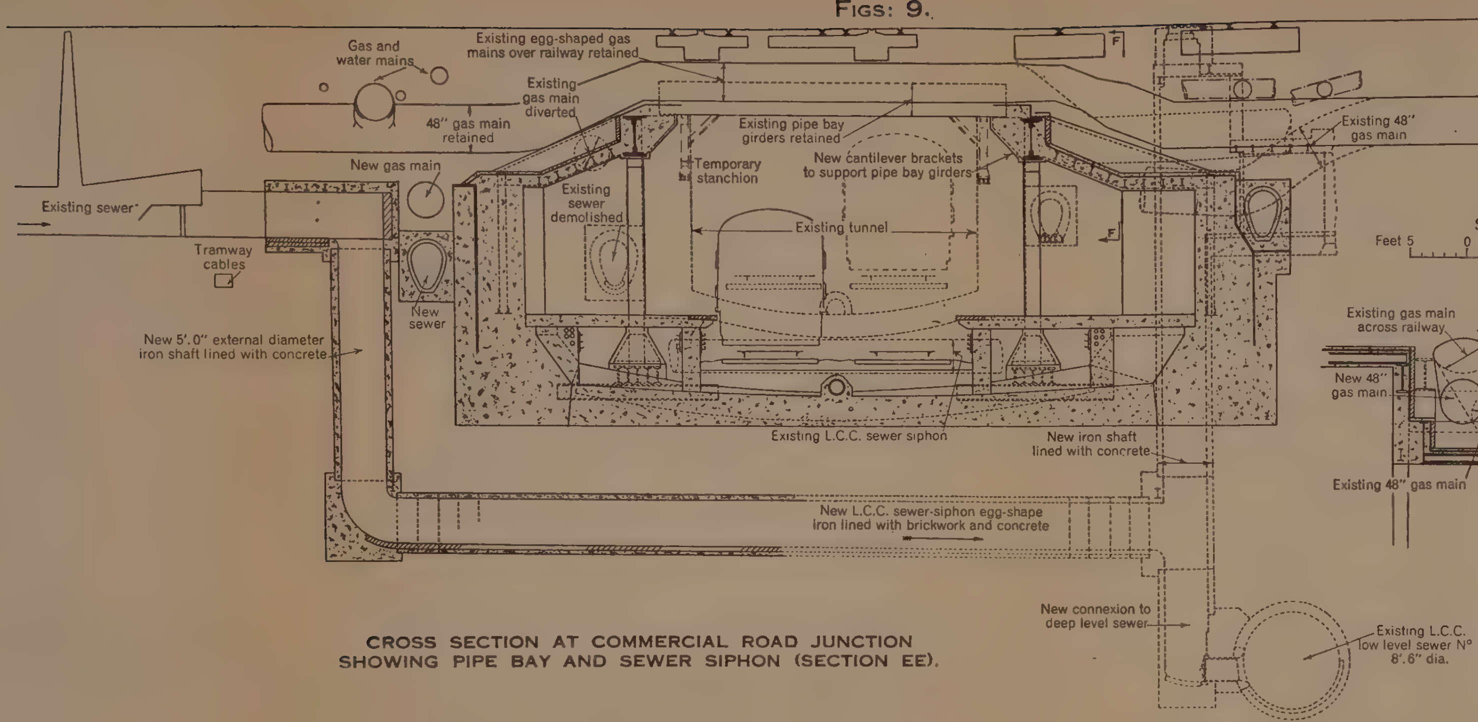
NEW ST

William Clowes & Sons, Limited: London.

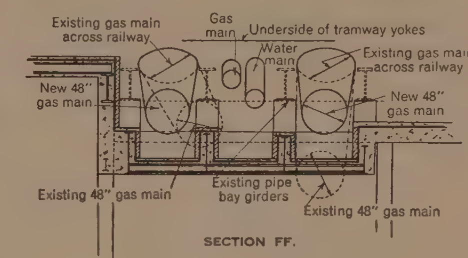
TATION.



Figs: 9.

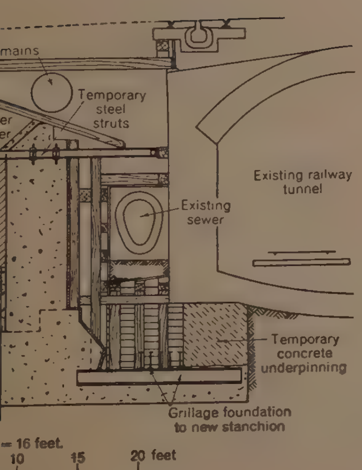


CROSS SECTION AT COMMERCIAL ROAD JUNCTION SHOWING PIPE BAY AND SEWER SIPHON (SECTION EE).



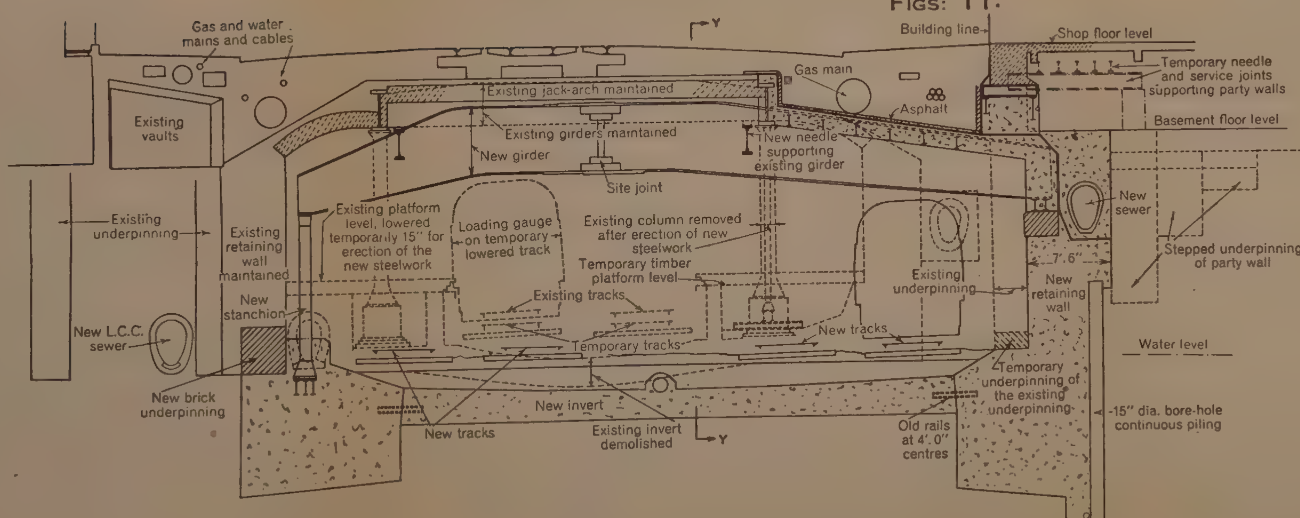
SECTION FF.

Fig: 8.

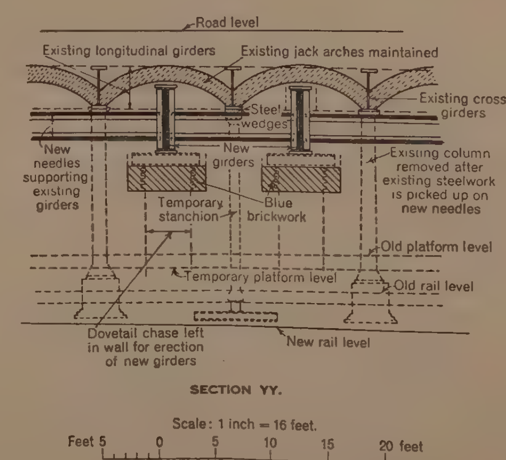


CTION OF NORTH WALL OF
ON DURING CONSTRUCTION
(SECTION DD).

Figs: 11.



CROSS SECTION THROUGH OLD STATION SITE (SECTION GG).



SECTION YY.

J. H. HARLEY-MASON.

JOINT MEETING.

10 March, 1939.

Sir HENRY PERCY MAYBURY, C.B.E., K.C.M.G., C.B.,
in the Chair.

A Joint Meeting, organized by the Institution of Automobile Engineers, was held in the Great Hall of The Institution of Civil Engineers with :—

The Diesel Engine Users' Association.
The Institute of British Carriage and Automobile Manufacturers.
The Institute of Fuel.
The Institute of Metals.
The Institute of the Motor Trade.
The Institute of Transport.
The Institution of Automobile Engineers.
The Institution of Electrical Engineers.
The Institution of Highway Engineers.
The Institution of Locomotive Engineers.
The Institution of Mechanical Engineers.
The Institution of the Rubber Industry.
The Iron and Steel Institute.
The Junior Institution of Engineers (Inc.).
The National "Safety First" Association.
The Omnibus Owners' Association.
The Royal Aeronautical Society.
The Tramways, Light Railways and Transport Association.

at which three Papers on "Comfort in Travel" [1] were presented :

Section I—By Road. S. E. Garcke, M.I.Mech.E., M.Inst.T.
Section II—By Rail. Rt. Hon. Lord Stamp, G.C.B.
Section III—By Air. Capt. E. W. Percival, M.I.Ae.E.

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[1] Journal I.A.E. vol. vii, No. 6, p. 17 (March 1939). The discussion will appear in an early number of the I.A.E. Journal, and the Papers with discussion in Proc. I.A.E., vol. xxxiii, to be issued about September 1939.

Section I—"By Road."

By Sidney Emile Garcke, M.I.Mech.E., M.Inst.T.

(*Abridged.*)

It has only been found practicable within the narrow limits of sp available to include a general survey of so wide a subject, and techni detail has had to be omitted. Travel on the road by private motor car a by public service vehicle are dealt with together, as, although the econom problems are quite distinct, many of the engineering features which rel to comfort are common.

Influence of Improved Road Surfaces.

It is interesting to observe that historically much of the improveme in comfort in travel by road can be attributed to the development of t supporting element used, namely, the surface of the highway. The st rail was comparatively smooth from the beginning, and the air is a verital cushion, although at times turbulent, and being, unlike the highway or t rail, a natural element, is incapable of improvement. The invention the railway was essentially the product of necessity, and the necessity the main was to find something better than the water-bound flint or sto road.

Until a little more than a century ago steam-propelled vehicl lumbered over these ill-kept roads. The constructors and operators these steam coaches were soon financially crippled mainly by the destru tive effect of excessive vibration, although legal restriction played its pa The invention of the steel railway marked the end of this particular pha of passenger transport, and the roads went into virtual disuse for near a century. Then Gottlieb Daimler invented his internal-combusti engine, little thinking that he was to be responsible thereby for not on revolutionizing transport, but at the same time bringing about a comple social change, preparing the way for an enormous increase in the efficien of the instruments with which men kill each other, and, in short, material influencing the course of civilization. Even as late as 1900, howeve Daimler's device remained but a hobby and toy. The point of interest that on the rare occasions in those early days when a motor-car pass over newly-laid smooth wood paving, the comfort was marked, and those old cars had been able to run on a road of the standard of t modern London to Brighton road, it would demonstrate the very lar part played by the improvement of the highway.

Definition of "Comfort" in Travel.

It is important to distinguish between what may be described as "mental comfort" and "bodily comfort," and, in regard to the former, to see that the would-be passenger has pleasurable anticipation rather than memories of discomfort. Mental comfort is contributed by such factors as the absence of uncertainty as to how the journey will be continued when changes of vehicle are necessary; the amenities of the vehicle itself, etc.

It has to be admitted that many of the 'bus companies are tending to adopt schemes of multi-coloured flamboyant ornamentation, both external to and within the vehicle, which give pleasure to the designer rather than to the public. The public comfort is to some extent affected by these matters, and the employment of professional artistic skill might please the public and save money by the removal of useless odds and ends intended to be "pretty." The passenger often has to be deceived, and, in fact, wishes to be. For instance, there is the seat which suggests luxurious lounging, but does not provide it. Again, there is the rich curtain down each pillar of the 'bus, which is not designed to be drawn across the window; it exists solely to give the would-be passenger a sense of luxury, while its only practical effect is substantially to reduce the field of vision and thus quite materially to contribute to discomfort. Experience shows that this reduction of physical convenience and comfort is more than compensated by the suggestion of a luxury comparable with the private limousine.

Factors Affecting Comfort.

Safety.—Probably a feeling of safety contributes as much as anything to that sense of well-being essential to travel comfort. The remarkably good record of safety to 'bus passengers—although, of course, the 'bus, like other road vehicles, contributes its quota of damage to third parties—when suitably advertised has the effect not only of encouraging 'bus travel, but subsequently of impressing on the mind of the passenger that the risk to him is negligible. Some of the figures of 'bus miles run in relation to harm to passengers are very striking, and one leading 'bus company recently advertised, in suitable pamphlet form, that in 20 years of public service 270 million passengers had been carried without killing one of them. Unfortunately, a serious accident involving the death of passengers occurred immediately on the issue of this piece of otherwise excellent propaganda.

Facilities for Rest, Food and Shelter.—These are important as a passenger by road cannot be comfortable if he is requiring rest and food. The 'bus or coach passenger, making anything but a very short journey, is rather in the position of a prisoner, but as the runs in Great Britain are relatively short, the necessity for the construction of special accommodation is not so obvious as on long continental journeys.

Silence, Suspension and Tires.—Though the general amenities ju
touched upon are of great significance in an analysis of comfort, they mu
naturally take second place to such physical matters as silence, smoo
roads, good tires, springs, seats, ventilation, and other factors contributi
to the smooth operation of the vehicle. The importance of the improv
ment in the roads and their bearing on the problem is obvious ; but t
improvement in tire characteristic, still continuing, is one of the mo
striking aspects of the matter. The cushioning capacity as between t
solid tire used on the immediate post-war 'bus and the latest tire now bei
supplied, is something of the order of a 500-per-cent. improvement, a
the corresponding figures for private cars as between the high-pressu
tire of a few years back and those in common use to-day show an advan
of approximately 33 per cent. The contribution to comfort in travel l
the greater resilience of the tire is not the complete story, for the reducti
in noise as the result of research in the matter of modern tread pattern h
assisted.

Design of Seats.—The design of seating to give true comfort for pa
sengers in private cars and 'buses necessitates most careful study. T
many seats have been devised on the assumption that passengers will
sitting in a stationary vehicle. Not only has the shape of the seat to l
designed so as to provide against forward movement and deceleratio
but there must be an approximate relationship between the springing
the cushion and that of the vehicle itself.

Heating and Ventilating.—Correct heating and ventilation of the vehic
must be placed high in the order of importance. Study of these questio
in relation to the latest 'bus designs has resulted in means being found bot
to ventilate and heat the vehicle adequately for the general condition
met with in this country. The problem is a personal one. Physic
characteristics and preferences differ widely. What seems to be necessa
for comfort is nothing less than individual control of the piece of atmosphe
surrounding the passenger, and that is far more difficult in a small transpo
unit such as a 'bus than in a train where this individual choice is obtaine
to a limited extent by compartmenting and by the provision of separat
smoking accommodation.

Factors to Combat Road Sickness.—Smooth acceleration, and, especiall
smooth deceleration, are important factors, as well as the elimination c
sharp corners and improper superelevation on roads.

Lighting and Visibility.—The question of artificial lighting in relatio
to comfort has reference almost exclusively to the public-service vehicl
because good lighting at night is of the greatest importance. To ensur
comfort a passenger must be able to read a newspaper under condition
of relative movement, which are rather greater than in other forms o
transport.

In order to improve interior illumination in daylight successful experi
ments have been carried out recently by introducing translucent, bu

opaque, synthetic resin panels in the roof of single-deck 'buses. In the case of both the private car and the 'bus of recent years there has been too great a tendency to reduce window size, resulting in a somewhat shut-in effect to the passenger. The comfort of the passenger has, undoubtedly, been sacrificed to the æsthetic aspect of pleasing lines.

One of the principal pleasures of the road adding to a passenger's comfort and the elimination of fatigue on long journeys is the ability to view the surrounding country without effort. Some of the latest coaches have the curved parts of the roof in glass, so that when touring in mountainous districts or sightseeing among high buildings a view can be obtained above as well as to the side. It is interesting to note that although the general introduction of low-loading levels for all 'buses has brought stability and improved appearance, these advantages have to an extent been neutralized by mechanical inaccessibility and the worsening of the view of the passenger when travelling through country with high hedges.

Comments on the Future.

Congestion of the Roads.—Road users appear to be moving in a circle in that the constant improvement of the highway has encouraged the greater use of motor vehicles. These, in turn, congest the roads and create a demand for still further improvements, and, incidentally, provide by taxation the means of the provision of such improvements.

Road-improvement schemes up to the present have been limited to the modernization of main arteries and other important roads outside urban districts for the obvious reason that the cost of such work in built-up areas has been almost prohibitive. The result is that the heavy additional traffic thus encouraged on the open country roads tends to congest the urban districts more and more, and the discomfort due to irritating delays on a journey in those districts is becoming more pronounced. If main-road development is to continue, then a very much larger outlay will have to be incurred on town route-improvement. The only alternative seems to be to limit the user of motor vehicles in the congested areas, but if that be done, then in all probability the further expansion in the use of motor vehicles on the road outside the cities will automatically be arrested.

Multiplicity of Signs and Signals.—It must be remembered that a large proportion of those travelling in private motor vehicles are drivers. The discomfort due to nervous reaction imposed upon a driver by the presence on the road not only of many signals, but of advertisements, lights, and other contrivances, readily confused with official signs, is very real. The use of such signs is now greatly on the increase, and will have to be controlled. Indeed, it might fairly be stated that even official signs, signals, warnings, and markings on the road surface are too many, and make a journey a test of nerves, even for the experienced driver.

Anxiety and confusion are also occasioned by the too-frequent use of

hand or illuminated signals on the motor vehicle. It is interesting to note that signalling by drivers is less frequent where driving is the most difficult, and from that it may be argued that danger and discomfort arise from the unskilled driver in the country who depends largely upon signalling in order to excuse an error in advance or counteract the effects of some fault already committed. At one time horn-blowing was the rule, but now this is officially discouraged. It is not an unreasonable supposition that with a raising of the average skill of driving, signalling, like horn-blowing, will be reserved for a few specific emergencies with a consequent increase in the safety as well as the comfort of road travel. While the Author deprecates too much signalling, and on the whole would prefer that cars should not be fitted with signalling apparatus, he nevertheless attaches importance to two pieces of apparatus which contribute to the ease and safety of driving. The one is the mirror and the other the red rear light connected with the brake.

The Future of Roads and Driving.—It would seem improbable that further marked improvement in road surfaces can be anticipated. The greater use of superelevation will help. More care in the designing and placing of notice boards and other signs, with possibly a reduction in number, may be expected, and will contribute to the comfort of the driver. Possibly the greatest improvement of all will come about automatically as a result of the gradual raising of the standard of driving skill.

Future Vehicle Design.—To anticipate marked further improvements in the vehicle itself requires boldness, because at least, so far as comfort is concerned, the automobile engineer has succeeded in making progress in a short time to a degree unrivalled by any other form of engineering development. It is probable that mechanical improvements will still be made which will effect economy, and a notable recent development in that direction is the general adoption of the compression-ignition engine for the heavier vehicle, but there is no reason why developments with an economic motive should bring in their train increased comfort, and it would seem that for improvement in this matter attention must be paid to factors outside the vehicle itself, and which relate rather more to the feeling of security and peace of mind for the passenger than to shielding his body from physical discomfort.

Section II—"By Rail."

By The Rt. Hon. LORD STAMP, G.C.B.

(*Abridged.*)

INTRODUCTION.

Careful consideration must be given, not only to the provision of physical comfort, but also to a variety of psychological factors such as freedom from strain and worry. These may be classified under the two headings "Physical Ease" and "Mental Ease," although, of course, as with nearly all classifications, there is some overlapping.

MENTAL EASE.

Preventive Measures.—Under this first head must be studied the traveller's feelings and economy of energies through each of several successive stages :—

(1) in finding out how to go;

(2) in finding out when to go;

(3) in giving him ease of access to the means of travel and of payment;

(4) in providing ease of disposal of his belongings;

(5) in giving ready assistance for whatever complications there must be on the journey;

(6) in freedom from anxiety about meals and other physical necessities;

(7) in the minimum duration of the journey and in its freedom from troublesome complications *en route*; and

(8) in a feeling of security and a mental background of safety.

Creative Amenities.—The following factors must be taken into account under this heading :—

(9) The conditions of the journey should make it easy for the traveller to conduct conversation (or not to be bothered by that of others); to be quiet and reserved, or to read (or perchance to sleep), or to observe the scenery, all according to his mood.

(10) He may be provided with special facilities for scenic observation and given folders, maps or guides for drawing attention to matters of interest. (This is particularly important for the tourist.)

(11) Sometimes provision of special accommodation for particular

classes of the community, " non-smokers," " ladies only," are helpf to secure the comfort of all concerned.

(12) Coming under this special category, of course, are many the amenities for night travel, and here the standard of public ex pectation has risen very rapidly. The provision of " gadgets " all kinds and of easy control of ventilation is of paramount importanc Freedom from disturbance by ticket collectors, customs officials, etc at night is important. The " morning cup of tea," which a few yea ago was a special amenity, is now a necessity.

Physical Ease.

This section will be considered in detail as follows :—

(1) absence of disturbing vibration ;
(2) absence of disturbing noise ;
(3) adequate heating and ventilation ;
(4) adequate lighting.

Absence of Disturbing Vibration.—Absence of disturbing vibrations i probably the most important factor in determining the physical comfor of passengers, and although the difficulty of securing it increases wit speed, great progress has been made in recent years. Smooth runnin depends both on the design of the rolling stock and track and also on thei maintenance.

Leaving for the moment the question of track, it is of the utmos importance that the wheels should move along it without violent latera movements which would transmit shocks to the vehicle and make ridin most uncomfortable. Recent investigations have shown that to secur steady forward movement without lateral shock depends fundamentall on the angle of coning of the tyres and on the subsequent maintenance o this correct angle. This problem has been studied theoretically by Dr F. W. Carter, Professor C. E. Inglis[1], and Dr. R. D. Davies[2], and botl in America and in Great Britain the motion of tires of different profile has been studied in practice by means of cinematograph records. Th conclusion has been reached both by theory and in practice that a profil as nearly cylindrical as possible gives the greatest freedom from high frequency lateral oscillations. A completely parallel profile is undesirabl owing to the need for counteracting unavoidable differences in diamete between the wheels at either end of an axle. An angle of 1 in 100 seem to be the best compromise, and this is now being widely used on the Londo Midland and Scottish Railway Company.

Another method of avoiding oscillation of the bogie is to mount th

[1] "The Vertical Path of a Wheel Moving Along a Railway Track." Journ Inst. C.E., vol. 11 (1938–39), p. 262. (March 1939.)

[2] "Some Experiments on the Lateral Oscillation of Railway Vehicles." *Ibid* vol. 11 (1938–39), p. 224. (March 1939.)

wheels so that they rotate independently of one another. This has been done in the "Duplex" bogie, one of which is running in Switzerland. In this way the tendency to bogie-hunting is entirely eliminated, there is no sliding motion when travelling round a curve, and the riding of the coach is excellent. The construction of the bogie is, however, much more complicated, and the cost is correspondingly greater. It is difficult to estimate how much rail- and tire-wear is reduced by smooth running, and to what extent the extra cost of the bogies would be met by the reduced maintenance and renewal costs of the permanent way, particularly as a large proportion of rail-wear is due to the driving wheels of locomotives.

In order to get accurate records of the riding properties of different types of construction and of their deterioration from wear and other causes, the Cambridge accelerometer is used to determine the transverse and vertical accelerations of a vehicle in motion. These are measured by means of the movement of two heavy weights held between stiff springs and free to move in a tranverse or vertical plane respectively, their movements being recorded on a strip of cellulose. The instrument is usually placed over the centre of a bogie in order to observe the maximum transverse oscillations of the vehicle. The advantages of getting definite measurements in this way instead of trusting to personal impressions are obvious, and the instrument has played a most useful part in the improvement of riding qualities.

To assist in the maintenance of the road in the best possible condition, two important aids have been instituted in recent years. The first of these is the use of the Hallade recorder, which makes a continuous record of the horizontal, vertical, and rolling movement of the vehicle in which it is placed as it passes over the line. These records are issued as charts to the inspectors and gangers on each section of the line, who can thereby locate points at which faults such as bad alignment, poor packing, and irregularity of level are beginning to develop. Great use, for example, was made of the Hallade method in improving the track between London and Glasgow for the accelerated timings of the "Coronation Scot," and as a consequence some two hundred and sixty-nine curves were re-aligned to promote smoother running or to relax or avoid speed restrictions.

Coupled with the necessity for accurate alignment and canting of curves, is the need for the maintenance of a true top to the track. The lack of true level can and does give rise to poor running at high speeds, and it is for the purpose of maintaining this true level that measured shovel packing has been introduced.

One of the difficulties in packing the sleepers is to determine the extent to which each sleeper requires attention. The apparent absence of line top can be ascertained relatively easily, but it is also necessary to measure the depressions which only occur under load. This is done by the use of voidmeters, which measure the depression of the sleepers when a train passes over them. By adding together the amount of visible and hidden

depression the quantity of additional ballast or chippings required t provide a firm bed under every sleeper can be calculated. This treatmer has been found not only to improve riding, but the track maintains it good condition for 50–80 per cent. longer than with the older methods c maintenance.

Absence of Disturbing Noise.—The main sources of noise are :—

(*a*) the rolling of the wheels on the rails ;
(*b*) impact at rail-joints ;
(*c*) rattling of brake-gear, movement of drawgear ;
(*d*) vibration and movements of the bodywork ;
(*e*) vibration of the steel panels due to air-flow ;
(*f*) whistling of the air past windows and ventilators.

There are four ways of reducing the noise-level in a compartment :—

diminishing as far as possible the production of noise ;
absorbing the noise at a point as near as possible to its source ;
avoiding the entry of the remaining noise into a vehicle ;
absorbing quickly the noise that enters.

Adequate Heating and Ventilation.—The normal body-temperature i regulated by the nervous system, which makes the heat-losses balance the heat which is generated by the chemical changes proceeding in the body. Comfort therefore depends primarily on getting conditions which wil just remove the steady output of heat of an individual at rest, and thi differs with individual idiosyncracies. The comfort zone can be definec in terms of the air-temperature, the proportion of radiant heat, and the movement of the air and its humidity, which jointly determine the total rate of loss of heat from the body. The following values have been found to satisfy average comfort conditions in Great Britain for most individuals :—

Relative humidity	50–60 per cent.
Rate of air movement	25–50 ft. per minute.
Ratio of radiant heat to convectional heat	About 1 : 1.

In Great Britain there is no need for a complete system which provides for cooling the incoming air in hot weather, and a simpler system of forced air ventilation is now installed in a few well-known trains as well as in sleeping cars. The tradition of the open window dies hard, but in these trains those who wish can now have the advantages of comfortable atmosphere with the minimum of noise and dirt.

It may be mentioned here that the modern form of sliding windows above the large window panels are designed to provide air movement without introducing dirt or draught, in that they act as extractors if not opened too wide.

Adequate Lighting.—Adequate and attractive lighting is an important amenity on journeys made during the hours of darkness.

Conclusion.

This Paper commenced with an abstract classification of all the factors in travel comfort from the moment a journey is imagined or desired, right through until it is achieved and a happy memory, and the Author has endeavoured to show that those mental " boxes " are not empty, but are filled with up-to-date experienced scientific arrangement and contrivance, and that these contents are being constantly overhauled.

Section III—By Air.

By Capt. Edgar Wikner Percival, M.I. Ae. E.

(*Abridged.*)

Introduction.

During the early days of the evolution of the flying machine, passenger comfort was probably the last thing to be considered. All efforts were concentrated on getting the machine to leave the ground and to support itself for flights of quite short duration. There was first the problem of designing lifting surfaces that would satisfactorily support a load in the air, together with a form of structure that would give the greatest strength and necessary rigidity for the least possible weight, at the same time allowing the machine to progress through the air at a satisfactory speed.

In parallel with the aeroplane, of course, was the development of the internal-combustion engine to a state of efficiency that would produce a reasonable power/weight ratio. Very great progress has been made in the development of the aero-engine and the aerodynamic efficiency of the aeroplane, so that a certain amount of weight can now be allocated to the physical comfort of passengers.

Until fairly recently aeroplanes were considered to be useful only for a certain hardy type of sportsman, for passengers who had a special and most urgent reason for transport from one place to another, or for purposes of war. From a purely commercial point of view, therefore, the very costly development of aerofoil sections, structures, engines, and airscrews was not carried forward as rapidly as it might have been had it received the amount of Government support that it warranted.

Noise.

Noise is one of the most important factors to be considered when studying the comfort of passengers. It is not sufficient merely to provide a passenger with a comfortably cushioned seat in an air-conditioned cabin. Noise and vibration must be reduced in order to bring travel by air at high speeds up to the highest possible standards.

Noise and vibration are inherently difficult to cope with due to the powerful engines required to give the performance demanded by the present-day airline operators. Because of weight and space limitations it is at present considered impracticable to build the necessary double-walled enclosures which would be most effective in insulating the passengers

and pilot's compartment from noise. It is also impracticable to apply the same methods of reducing noises at their sources that are applied where weight is not an important limitation. Efficient engine mufflers which can readily be applied in automobiles would cause an excessive loss of power and payload, according to present-day standards, if used in aeroplanes. Even if this were not true, exhaust mufflers are of little value in aircraft, where the predominant source of noise is the airscrew.

Unlike the problems of the road vehicle, the designer of aircraft has the additional noise complication of the airscrew, which contributes the major portion of the total noise to be dealt with.

It can, therefore, be seen that the methods of reducing noise and vibration as applied to aeroplane construction must of necessity differ from the methods used in automobiles and other ground installations where weight is not an important factor.

Dr. A. H. Davies, of the National Physical Laboratory, Teddington, in his book "Modern Acoustics" states :—

> "In connexion with the suppression of noise at its source, it must be realized that if several sources of different loudness exist together, no appreciable improvement can be attained by suppressing any but the loudest. When that is reduced, the next loudest dominates, and must be suppressed in turn.
>
> "A useful procedure in noise investigation is therefore to ascertain which causes are responsible for the loudest components of the noise under study."

In a practical investigation into the noises in an aeroplane instruments must be used so that the main source can be attacked first. The instrument for carrying out this work is the "Analyser," which is capable of measuring the different components of the total noise and of giving the actual value of each.

When the best practical results have been achieved with regard to reducing the noise at the source, any further improvements must be achieved by treating the cabin itself. It must, of course, be remembered that the main source of noise must be treated first.

Although the Analyser is essential, it can only give an indication as to which noise is the most disturbing, but cannot give an indication as to the source of the noise. Therefore a directional microphone is of great value in ascertaining the location of the noise, and so by a process of elimination the machine in question will be rendered as silent as is practicable with the particular type.

In reducing noise in an aeroplane, the cabin of which has not been designed in collaboration with an acoustical engineer, the first problem is to reduce the resonating value of any panels or structural members that lie in the path of greatest intensity of the noise emanating from the airscrew. After this comes the question of absorption of the noise. Absorb-

ing materials are used to prevent this reflexion of the sound waves, and reduction of 10 decibels can be achieved quite practicably by these mean

A point worthy of consideration in multi-engined aircraft is the heter dyning effect due to the engines not being in perfect synchronism. Con plete synchronism can be achieved only if a synchronizing apparatus used, and these at the moment are not extensively used, but probably th expense would be justified.

Ventilation.

Soundproofing is not the only factor in the comfort of a commerci aeroplane; good ventilation is equally important. The necessity for bigger supply of air than that previously adopted has been brought t light by American statistics.

While crossing the Rocky Mountains, where the machines have t climb very steeply from aerodromes near the foot of the mountains, nearl 60 per cent. of the passengers were ill in the Fokker F.VII and For machines of 1932, in which the importance of ventilation had not bee realized. This percentage is now reduced to 2 since the adoption Douglas and Boeing machines, which are equipped with the very late form of air-conditioning. The physiological effect of steep climbs can b prevented by these means.

A minimum rate of air of 12 cubic feet per minute per passenger wa determined by medical advisers who studied the arterial pressure and th reactions of twelve passengers, six men and six women of different age while the supply of air was gradually varied in a modern aeroplane. It i recommended to increase the rate to 20 cubic feet per minute to reduc illness in bad weather and in the tropics.

With large aircraft there is little sense of motion, and adverse weathe conditions generally do not produce pitching and rolling. The pilots ar instructed to fly at a height which gives the greatest degree of comfor to the passengers, and often they are able to avoid storms altogether For those passengers who are bad travellers, various remedies are carrie (cotton wool and chewing gum) which can be obtained free of charge As a guarantee against illness, some passengers find glucose D and barle sugar very effective. The former palliative is particularly effective if th dose is taken an hour or so before the flight commences.

In introducing the air into the cabin, the speed should be less tha 15 miles per hour, to avoid the discomfort of a high-speed air current an whistling noises.

It is very much more effective to have the complete cabin correctl air-conditioned, expecially in very large aircraft, than to use the ol method of individual ventilators by each passenger's seat. The individua passenger ventilators, from which the unitary supply is small, are quit insufficient to deal satisfactory with the volume of air required for th

complete cabin, without increasing the velocity beyond that which is desired.

Heating.

Heating cannot very well be separated from ventilation, as together they constitute a complete system of air-conditioning. The quantity of heat necessary to keep the temperature at the desired value in large aeroplanes is a problem which has not yet been completely solved.

There are several methods in use, the majority using the exhaust from the engine as the heating element. In some cases, the air is taken from an annular duct on the exhaust manifold, whilst another system is in the form of a steam boiler surrounding the exhaust pipe, which circulates hot water to baffles in the air-duct, and the air passing over these baffles is thus heated and supplied to the cabin in a warmed condition.

Another form of heating is by means of electrically-heated elements through which air is drawn and introduced to the cabin by means of ducts.

Seating, Visibility, and Interior Layout.

The question of the passengers' seating accommodation has received a very great deal of attention during the past few years. The seating arrangements in the modern air liner are the most comfortable that can be found, and in the majority of large machines the individual seats are adjustable from a vertical position to fully reclining, by the pressure of a lever. These special chairs are so designed that, as the reclining position is taken, the seat is gently lowered, thus leaving the occupant with his feet comfortably on the deck, in contrast to the old type of tilting chair, where only the back was hinged. Lap straps are provided on the chairs. Their use is optional, but some passengers find they increase the feeling of steadiness and comfort. It is recommended that they be used in bumpy weather and during take-off and landing.

Conclusion.

The Author considers that it can be said without fear of contradiction that the development of the aeroplane has progressed at a very much greater rate than any other form of transport vehicle. Whilst, however, those concerned with development are striving for increased performance, little has been done to reduce the time taken to transport passengers and goods between aerodromes and the town or city centres which they serve.

Means should be used by local authorities to provide a more direct route. Probably the most satisfactory, and cheapest in the long run, would be elevated electric railways, transporting passengers non-stop between town centres and airports; during the journey Customs examinations

and other necessary formalities could be carried out. The Author is qui sure that this form of connexion would provide a much more satisfacto solution to the problem than proposals previously put forward, such aerodromes spanning rivers in the centres of cities, or platforms on t tops of tall buildings.

This form of transport would be rapid and would allow airports to placed on the outskirts of the populated areas which are usually fair free from fog, and where land is a great deal cheaper to acquire and easi to prepare ; this latter point is an important consideration.

For very large cities it will be essential to have several main aerodrom in order to take care of the ever-increasing traffic, especially from the poi of view of operating in conditions of bad visibility, where, at a crowd airport, aircraft must wait their turn to land. The elevated electr railway system would radiate from a common centre, if necessary, to t various airports, or, in any case, from points within easy reach of t centre of the city, and would add greatly to the high standard of comfo that is to-day provided in air travel.

Paper No. 5169.

"The Strengthening and Final Testing of the Pressure Tunnel for the Water-Supply of Sydney, N.S.W."

By Samuel Thomas Farnsworth, B.Sc. (Eng.), M. Inst. C.E.

(*Abridged.*)[1]

(*Ordered by the Council to be published with written discussion.*)[2]

TABLE OF CONTENTS.

Introduction.

The purpose of this Paper is to complete the record of investigations and work carried out subsequent to the action described in a previous Paper by Mr. Gerald Haskins, M. Inst. C.E.[3] The latter Paper describes the original proposal for twin 7-foot diameter pressure tunnels to convey water to the city of Sydney, the construction of the subsequently-approved 10-foot diameter concrete-lined tunnel, its testing and failure, and the methods decided upon to strengthen the tunnel by concreting-in 8-foot 3-inch diameter mild-steel bitumen-lined tubes to form an impervious lining. The remedial work was suspended in March 1931, but was resumed

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[1] The MS. and illustrations may be seen in the Institution Library.—Sec. Inst. C.E.

[2] Correspondence on this Paper can be accepted until the 15th July, 1939, and will be published in the Institution Journal for October 1939.—Sec. Inst. C.E.

[3] "The Construction, Testing and Strengthening of a Pressure Tunnel for the Water-Supply of Sydney, N.S.W." Minutes of Proceedings Inst. C.E., vol. 234 (1931–32, Part 2), p. 25.

in August 1932; section 1 of the tunnel was completed in November 193 and section 2 in September 1935.

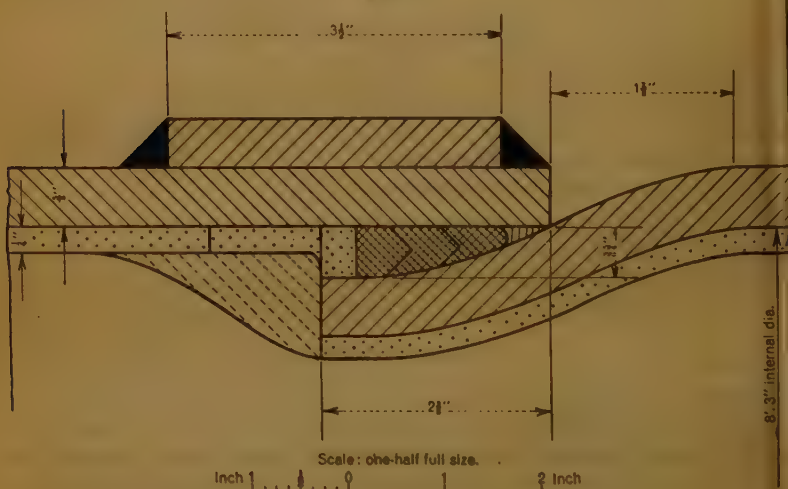
The tunnel is 10 miles long and is in two sections at different leve There were seventeen construction shafts, of which eight were used offtakes, whilst two others were used for unwatering the tunnel. Th tunnel was lined with a bituminous material having the following com position:

" R_2 Mexphalte ", with 30/40 penetration .	90 per cent.
Diatomaceous earth	10 per cent.

Tunnel-Joints.

The type of internal socket-joint used for the tunnel-pipes is shown *Fig. 1.* The joints were made with special extruded lead sections, su

Fig. 1.



sequently hand-coated with bitumen and finished to a smooth faire surface. The special lead sections, inserted in rotation after the initi sealing ring of rubber, were caulked with specially-shaped caulking tool to give an hydraulic-leather effect to the V-shaped back of the previo lead section. Electrically-heated screeds were used to give the fin shape required.

Closing-Pipes, Shaft-Lining, and Offtake-Details.

The closing lengths were enveloping pipes with socket-joints also ma with extruded lead, bitumen-coated and faired.

The three methods of shaft-lining used were (*a*) normal internal socket-joints, bitumen-lined, (*b*) internal-angle flanged joints, cement-lined, and (*c*) external-angle flanged joints, bitumen-lined. The length of lining pipes was originally 9 feet, but this was increased to 12 feet owing to the change in conditions of supply of plates.

The original design provided for vertical needle valves set at rock-level to control offtakes. The final installation provides for horizontal needle valves at the surface of each offtake-shaft except the final offtake-shaft (No. 17), where both vertical and horizontal valves are installed. The other control-equipment at the offtake-shafts comprises a geared 48-inch gate-valve to each branch, a 12-inch relief-valve and stop-valve control, a 4-inch air-valve with control-valve, and an expansion-joint with an insulated joint and an electrical bond.

Other Work.

Dewatering Arrangements.

Provision for unwatering the tunnel is made at two shafts, Nos. 5 and 11, each of which is equipped with a two-stage centrifugal pump having a capacity of 2,100 gallons per hour, discharging to a stormwater-channel by a 12-inch diameter pipe. It is intended to use these two shafts as offtake-shafts in the future, and a 30-inch diameter cast-iron flange-pipe connected by a branch-piece into the invert of the tunnel is provided at each shaft. These offtake-pipes are controlled by needle valves.

Groundwater Drainage-System.

Groundwater drains were provided above and below the tunnel-lining, but no circumferential connexion is made between the two, experience indicating that there is no necessity to do so. The upper drains in both sections are connected together at shaft No. 6, where the step-up between the two sections of the tunnel occurs. Arrangements for sealing or emptying this drain are made at shafts Nos. 5 and 11, so that the groundwater-pressure may build up when in service and may be reduced to zero when the tunnel is empty.

Electrical Bonding and other Details.

For some 40 feet at the foot of shaft No. 1 (the intake-shaft), the lining is of reinforced concrete only. A water cushion is provided at the bottom of the shaft. At the commencement of the bitumen-lined pipes in the tunnel an octagonal steel diaphragm measuring 17 feet by 17 feet by $\frac{5}{16}$ inch was mounted. This was to prevent the percolation of tunnel-water to the groundwater drain. This reinforced-concrete section forms a break in

the electrical continuity of the steel lining, and four 5-inch by ½-inch mi[illegible] steel flats were fitted to make electrical connexion between the lining of t[illegible] shaft and that of the tunnel. A similar bond was needed at shaft No. [illegible] where the new lining and the original are not in actual contact.

Cast-Steel Bulkhead.

To permit the testing of the tunnel in two sections a cast-steel reversib[illegible] bulkhead was constructed adjacent to shaft No. 6 in the upper section of t[illegible] tunnel.

Testing the First Section of the Relined Tunnel.

The first section of the work was completed by November 1933, a[illegible] was immediately tested.

Testing Committee.

A testing committee was appointed to supervise the testing of th[illegible] tunnel. The committee consisted of Messrs. H. H. Dare, M.E., A. [illegible] Debenham, B.E., and Gerald Haskins, MM. Inst. C.E., Professor W. [illegible] Miller, M.E., B.Sc. Assoc. M. Inst. C.E., the President, Mr. T. B. Coope[illegible] and the Author.

General Arrangements.

The first section of the tunnel was tested under heads of 110 feet, 2[illegible] feet, 310 feet, and 400 feet of water. For the tests under the first thr[illegible] heads, the water-level was below the cap at shaft No. 1 and the loss [illegible] leakage was measured by means of gauge-boards mounted on the side [illegible] the shaft. For the 400-foot test the water-losses were measured by mete[illegible] with a check measurement of the leakage using a 12-inch standpipe.

The meter used was a 6-inch inferential type, with a 1-inch bypa[illegible] positive meter for slow rates of flow. A 1-inch crown meter was installe[illegible] at shaft No. 5 in the groundwater-system so that groundwater-flow coul[illegible] be measured if necessary. Provision was made to measure any leaka[illegible] at the bulkhead. The temperature of the inflowing water and of the wat[illegible] in the tunnel at each shaft was recorded.

A final inspection was made and the first section of the tunnel w[illegible] found satisfactory. Filling of the tunnel (first section) commenced on t[illegible] 30th January, 1934, and was at the rate of 60,000 gallons per hour.

Application of a 110-foot Head.

Raising the head from zero to 110 feet was carried out at a steady ra[illegible]

over 24 hours. The initial leakage was at the rate of 48 gallons per hour. This head was maintained for 7 days.

Application of a 210-foot Head.

Further filling was carried out over a period of 24 hours. The initial gross leakage was 150·3 gallons per hour, but as there was a leakage at the bulkhead at the rate of 61·5 gallons per hour the initial net leakage was at the rate of 88·8 gallons per hour.

After a leakage which was discovered at the needle-valve in shaft No. 5 was repaired the net loss dropped to 6·1 gallons per hour. This head was maintained for 12 days.

Application of a 310-foot Head.

The initial leakage was 16·2 gallons per hour, which dropped after 12 days to 8·3 gallons per hour.

Application of a 400-foot Head.

Filling to this head necessitated the fitting of cap-covers on all the shafts, and in so doing care was taken to release all air and to avoid shock. After caps had been fitted final filling was carried out through a 1-inch bypass. Observations at this head were continued for 7 weeks. Initial leakage was at the rate of 41·5 gallons per hour, but the final figure was 18·75 gallons per hour. To check the figure of 41·5 gallons per hour a 12-inch-diameter standpipe was erected at shaft No. 1 and the loss was measured by direct observation of the water-level in the pipe. The results so obtained gave an indicated loss less than that actually measured at the bulkhead and needle valve. The apparent discrepancy is due to the compressibility of such a large volume of water. The average leakage was at the rate of 25 gallons per hour over the whole period, the lowest recorded rate being 6·1 gallons per hour.

The result of the test was considered satisfactory by the Committee.

Testing the Second Section of the Relined Tunnel.

The second section was completed in September 1935 and it was tested in a similar manner to the first section. The bulkhead at shaft No. 6 was reversed to isolate the second section.

Testing Committee.

The special testing committee was reconstituted with the same personnel

as before, with the exception that Mr. T. H. Upton, O.B.E., M.Sc. M.C.E., M. Inst. C.E., had succeeded Mr. T. B. Cooper as President.

The Filling of the Tunnel.

Filling commenced on the 3rd September, 1935, at the rate of 48,00 gallons per hour. Provision was made at the bulkhead to allow the escap of entrapped air between shaft No. 9 and the bulkhead.

Application of a 93-foot Head.

The filling was carried out at a regular rate during a period of 24 hour and was maintained for 8 days. The initial leakage was 10·5 gallons pe hour, and the final figure was 0·24 gallon per hour.

Application of a 186-foot Head.

The initial leakage was at the rate of 42 gallons per hour, whic decreased in 5 days to 7 gallons per hour.

Application of a 285-foot Head.

The top water-level for this test was above the caps of the shafts an had to be applied from elevated tanks. The initial rate of leakage wa 33 gallons per hour, which diminished in 4 days to 11 gallons per hour.

Application of a 315-foot Head.

An extra 30-foot head was obtained by raising the tanks. The ne initial and final leakage-rates were 25 and 11 gallons per hour respectivel

Throughout these tests the groundwater pressure was kept at zero.

Inspection of Second Section: Blistering of Bitumen Lining.

The tunnel was unwatered after completion of the tests, and inspectio showed that the whole of the lining, including the joints, was in perfe condition, with the exception of a strip about 2 feet wide along the soffit the pipe, where blistering had occurred. This was apparently due absorption of air under pressure causing blisters when the pressure w released. Water marks were observed in the blistered area, apparent corresponding to the successive water-levels.

Confirmatory Experiments.

Experiments have since been carried out to investigate the absorption of air by bitumen. Blisters similar to those which occurred in the tunnel have been obtained.

The Committee's Report.

The final report declared the finished tunnel to be entirely satisfactory.

The results of the tests on the two sections may be summarized as follows:

Section 1: leakage 1·76 gallon per 24 hours per inch diameter per mile, or 0·011 gallon per 24 hours per linear foot of field-joint.

Section 2: leakage 0·346 gallon per 24 hours per inch diameter per mile, or 0·003 gallon per 24 hours per linear foot of field-joint.

The Paper includes a Table comparing the leakage of the Sydney water-supply pressure tunnel with mild-steel pipe-lines of which figures are obtainable.

Condition of Tunnel after Service.

The tunnel was placed in commission on the 4th November, 1935, and was in service throughout the summer months. Towards the end of April 1936 action was taken to unwater and inspect the tunnel. The air blisters were found to have flattened out, but slight water blistering was found in the bitumen lining of the cast-steel thrust-ring on which the bulkhead had been mounted. Patches of fresh-water *polyzoa* were noticed.

The tunnel was recommissioned in June 1936, and it is not intended again to inspect the tunnel until 1942.

HYDRAULIC-FLOW TESTS.

Value of Williams–Hazen Coefficient.

During discussion of the remedial measures in progress in 1933, the question arose regarding the true loss of head that would occur under working conditions. In determining the design the Williams–Hazen formula had been used. This gives the relation $Q = AV = ACR^{0 \cdot 63} s^{0 \cdot 54} 0{\cdot}001^{0 \cdot 04}$, where Q denotes the flow in cusecs, A the cross-sectional area in square feet, V the velocity in feet per second, r the hydraulic mean radius in feet, and s the hydraulic gradient in feet per foot, whilst C is a coefficient. It was believed that the value of C for a smooth-bore spun bitumen-lined pipe should be about 140. A value of 120 was, however, taken, which it was anticipated would allow satisfactorily for additional losses due to

the protuberances of the internal joints, to bends, to entrance turbulence, etc. This assumption gave a figure of 20 feet for the total loss of head with a flow of 100,000,000 gallons per day.

The opinion was expressed by Mr. T. D. J. Leech, B.Sc., B.E., that the loss of head due to the internal joints might not be sufficiently covered by the assumption of the value of 120 for the Williams–Hazen coefficient, but that without experimental data he was not prepared to recommend a definite value for the coefficent. The Water Board accordingly provided funds to permit experimental work to be carried out on scale models of the pressure-tunnel pipe sections, and Mr. Leech made the necessary tests. An abbreviated copy of Mr. Leech's report accompanies the Paper.

Scale-Model Tests.

Cast-iron models were made of the steel pipes, reproducing the contours of the streamlined joints. For reasons given in Mr. Leech's report the scale adopted was 1 to 16·1. By means of a special tilting micro-manometer, the loss of head across plain pipes and across pipes with internal joints was obtained. The loss in the latter case was shown to be about 3 times that in the former case. From a comparison of these experimental results with the assumptions made in the designs for the remedial measures, it was concluded that the loss of head would be approximately 1·75 times greater than that originally calculated. For a flow of 100,000,000 gallons per day this meant a total loss of head over the whole length of tunnel of 35 feet instead of the estimated 20 feet, the corresponding values of the Williams–Hazen coefficient being 88 and 120 respectively. In these tests and calculations it was assumed that 9-foot lengths of pipe would be used throughout for the construction of the tunnel.

It was decided to carry out full-scale flow-tests when the tunnel was completed, in order to verify the figures forecast by the model-tests.

Full-Scale Flow-Tests with Tunnel in Operation.

In view of the length and diameter of the tunnel, and of the necessity for extreme accuracy in the measurement of pressures, special mercury-column gauges were constructed and were mounted at shafts Nos. 1, 15, and 17. The gauge installed at shaft No. 15 was provided to serve as a check on the results obtained at shaft No. 17.

Sufficient height was provided in the mercury columns to balance the static pressure due to the water-level in Potts Hill reservoir and thus to register on the scale attachment, graduated in inches and tenths, the drop in pressure under flow-conditions. Zero levels in the mercury pots, against which the water pressure acted, was maintained by an adjustable

regulator screw on the principle of the Fortin barometer. The temperature of the mercury was recorded to enable an accurate determination of the height of the equivalent column of water, whilst the temperature of the water entering and leaving the tunnel was noted both before and after the test period as a factor in the computation of the Reynolds number.

The flow-measurement was obtained by means of the Venturi-meter permanently installed in the connexion from Potts Hill reservoir to shaft No. 1 of the tunnel. Water manometers connected directly to the throat and upstream of the meter were used instead of the usual mercury-type manometers, and were so located that the tops were open to the atmosphere. Thus under no-flow conditions the level in each tube was the same, and coincided with that of the water in Potts Hill reservoir.

The differential pressure-head, under flow-conditions, was observed against a scale attachment, graduated in inches and tenths, from which the flow was computed in accordance with the Venturi law

$$h = \frac{V_2^2 - V_1^2}{2g} \times \frac{1}{c^2}.$$

The probable error of observation was not greater than 0·25 inch of differential head, corresponding to an error not greater than 1 per cent. at the maximum flow obtained.

The test was carried out over a 3-hour period, commencing with no flow, so that a common datum at each of the mercury pressure-gauges could be established. The flow was then increased by increments according to a predetermined programme to a maximum feasible discharge of 47,500,000 gallons per day.

Observers were stationed at the Venturi-meter and at each of the mercury gauges, and observations were recorded during the test period at 60-second intervals, all watches having been synchronized before commencement of the test.

The loss of head due to friction over the distance of 51,949 feet was 7·32 feet for a flow of 47,500,000 gallons per day. The loss of head per 1,000 feet was 0·141 foot, giving a value of C in the Williams–Hazen formula of 96.

Thus a comparison of the originally assumed, experimentally forecasted, and final, friction-losses over the full length of tunnel from shaft No. 1 to shaft No. 17 can be indicated as shown in the Table on p. 570.

When the fact is taken into account that the model-tests were carried out on the assumption that the pipe sections were in 9-foot lengths and that in actual fact 56 per cent. of the total number of sections are 12-foot lengths, the figures forecasted by the model-experiment and those of the actual test compare very favourably.

Flow: gallons per day.	Total friction-head loss: feet.		
	C = 120 (original assumption).	C = 88 (model-experiment forecast).	C = 96 (final test of completed tunnel).
47,500,000	4·8	8·6	7·3
100,000,000	19·2	34·1	29·0

Cost of Remedial Measures.

The estimated cost of the remedial measures was £924,000, as set out in the previous Paper[1]. The actual completed cost was £854,000.

Conclusions.

Behaviour of the Tunnel.

When it is considered that the length of the pressure tunnel is 10 miles with circumferential lead joints at intervals of 9 feet and 12 feet throughout this length, the extraordinarily low leakage is a matter for great satisfaction. Undoubtedly this result must be attributed to the form of joint used, and to the care with which the work was carried out.

Behaviour of the Bitumen Lining.

The blistering in the air space during testing and the confirmatory laboratory results furnish a new aspect on bitumen linings, and certainly indicate a degree of porosity, or capacity for absorption of fluids under pressure, which had not previously been anticipated. In particular, the granulation of the lining material indicates a physical action, the precise mechanics of which have not yet been discovered. Pressure of water alone on the sides of the pipe could not produce the granulation observed in the air-space zone of the lining. Since moisture was present in the blisters produced below the water-line in the laboratory tests, it may be concluded that water vapour must accompany the air which causes the blistering, but it still remains to be determined whether this moisture can cause damaging corrosion under the bitumen.

The high-pressure air test has since been applied to a number of bituminous pipe-lining mixtures, and markedly varying results have been obtained. As a test for suitability of linings the method has promise, although ageing and other phenomena are not covered.

[1] Footnote ([3]), p. 561, *ante*.

Use of Scale Models.

The very satisfactory results obtained on the determination of friction-losses by the use of scale models is a striking instance of the value of model-tests in civil engineering. In the case of the internal joints of the tunnel there is an illustration of the difference between surface-roughness effects and the effect of change of form. Calculated as a succession of contractions and expansions by the usual formula, the effect of the joints appears to be negligible; actually, however, in the case under consideration, the relative dimensions are such that the projections are not changes in form in the hydraulic sense, but may be regarded as increasing the general turbulence by changing the boundary-layer conditions.

It should be mentioned that the actual increase in head-loss from 19·2 to 29·0 feet for a flow of 100,000,000 gallons per day, demonstrated by the models and confirmed by test, is of comparatively little practical consequence, since it can readily be compensated for by a slight increase in the height of the proposed elevated reservoir at Potts Hill whenever this is constructed.

Acknowledgements.

The Sydney pressure tunnel, despite its failure as originally constructed, is now undoubtedly a most efficient unit of the Sydney water-supply distribution-system, and the Author desires to repeat the acknowledgements of the most valuable assistance given by Messrs. H. H. Dare and E. G. Ritchie, MM. Inst. C.E., made by Mr. Gerald Haskins, M. Inst. C.E., in the previous Paper[1], in the conception and design of the very successful remedial measures.

Mr. W. D. Goudie acted as Inspecting Engineer and Mr. D. G. Bruce as Resident Engineer throughout the period of operations described in this Paper. Mr. T. B. Nicol, B.E., was Designing Engineer.

Most valuable assistance was given by Mr. T. D. J. Leech, B.Sc., B.E., Lecturer in Civil Engineering at the University of Sydney, in devising and carrying out the scale-model tests on hydraulic characteristics of the tunnel, and the Author is indebted to Mr. Leech for permission to include a copy of his report on the tests as an Appendix to the Paper.

Mr. Cyril Spooner, Assoc. M. Inst. C.E., as Investigating Engineer, conducted the flow-test experiments on the completed tunnel, and supervised the hydraulic calculations.

Mr. F. de L. Venables, B.E., Research Engineer, gave assistance during the testing operations, particularly in research on the behaviour of bitumen under compressed air. Mr. Venables also gave assistance in the preparation of this Paper.

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[1] Footnote ([3]), p. 561, *ante*.

The Author is indebted to the President, Mr. T. H. Upton, O.B.E., M.Sc., M.C.E., M. Inst. C.E., and the members of the Metropolitan Water Sewerage and Drainage Board, for permission to present this Paper.

The Paper is accompanied by twelve sheets of drawings, from one of which the Figure in the text has been prepared, and by eight photographs and five Appendixes.

Paper No. 5209.

"Constructional Work on Air Raid Shelters and Other Protective Works."

By Robert William Goodwin Clerke, Assoc. M. Inst. C.E.

(*Abridged.*) [1]

(*Ordered by the Council to be published with written discussion.*) [2]

TABLE OF CONTENTS.

Introduction.

The works described have either been constructed or are under construction and are applicable to a wide variety of site and other conditions. Whilst not exhaustive, they represent the application of Home Office and other A.R.P. recommendations.

Protection from direct hits from high-explosive bombs do not come under consideration.

The works cover:

1. Protection of personnel from:
   - (*a*) Gas and incendiary bombs.
   - (*b*) Light explosive bombs.
   - (*c*) Impact and penetration from (*a*) and (*b*).
   - (*d*) Blast and fragmentation.
   - (*e*) Demolition.
   - (*f*) Concussion.

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[1] The complete MS. containing, in addition, some notes on design data, types of bombs, and recommendations for protection (from various sources) may be seen in the Institution Library.—Sec. Inst. C.E.

[2] Correspondence on this Paper can be accepted until the 15th July, 1939, and will be published in the Institution Journal for October 1939.—Sec. Inst. C.E.

2. Protection of power-plant from:
   - (a) Incendiary bombs.
   - (b) Light explosive bombs.
   - (c) Impact and penetration from (a).
   - (d) Blast and fragmentation.

Description of Works.

As the protection of plant at power-stations and other works provide for comparatively simple protective measures, these will be described first proceeding in stages to the more complicated shelters for personnel.

Fireproof catch-roofs.

To provide a means of protection for boilers and power-plant agains incendiary bombs, fireproof roofs are provided. A typical example i shown in Figs. 1, Plate 1, which shows a roof over a bank of boilers.

The design is based on the following lines:

- (a) As protection against 1- and possibly 2-kilogramme bombs.
- (b) The existing slate roof to act as an initial impact-sheet.
- (c) A secondary impact-sheet and a fireproof sheet, as shown, o asbestos cement.
- (d) An impact allowance of 20–30 lb. per square foot on the roof i also provided.

As is generally found to be the case, the existing roof-trusses are no strong enough to carry the whole of the additional load of the new roo if slung from them. The weight is, therefore, carried almost entirely o steel compounds carried down to the boiler settings.

The roof is given a decided slope to permit a bomb to roll off it on t the concrete floor below, where means of dealing with it with sand or foame slag are provided.

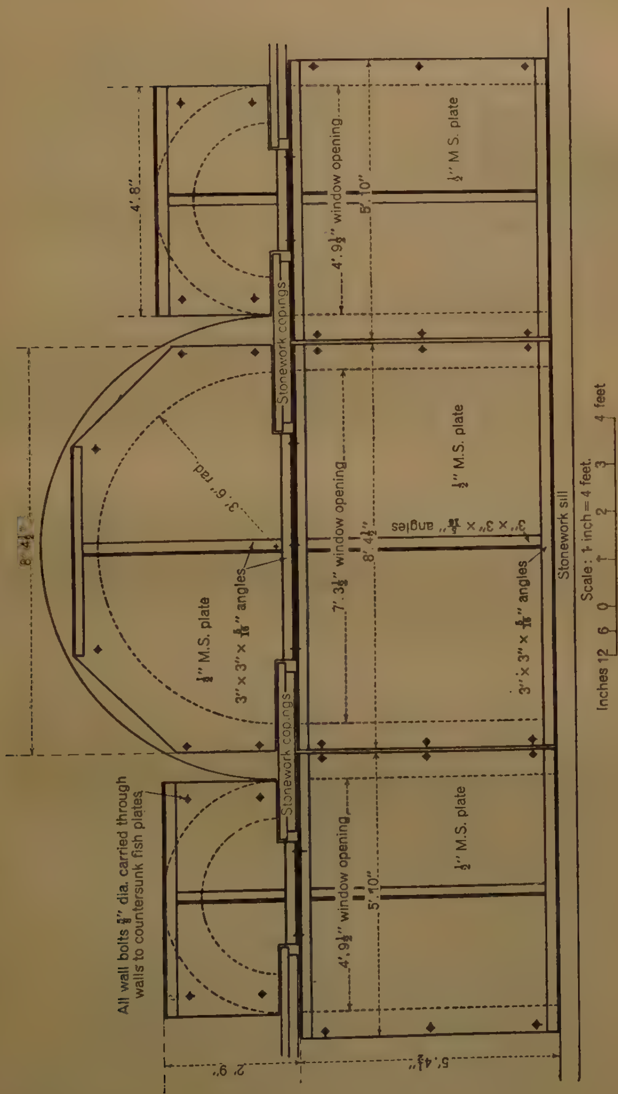
Steel shutters over windows.

Two types of steel shutters over large window-openings have bee evolved to guard against blast and fragmentation.

The shutters, as shown in *Fig. 2*, are used where the window opening are narrow or where pipes or other protuberances along the walls, insid the buildings, prohibit the fitting of shutters inside.

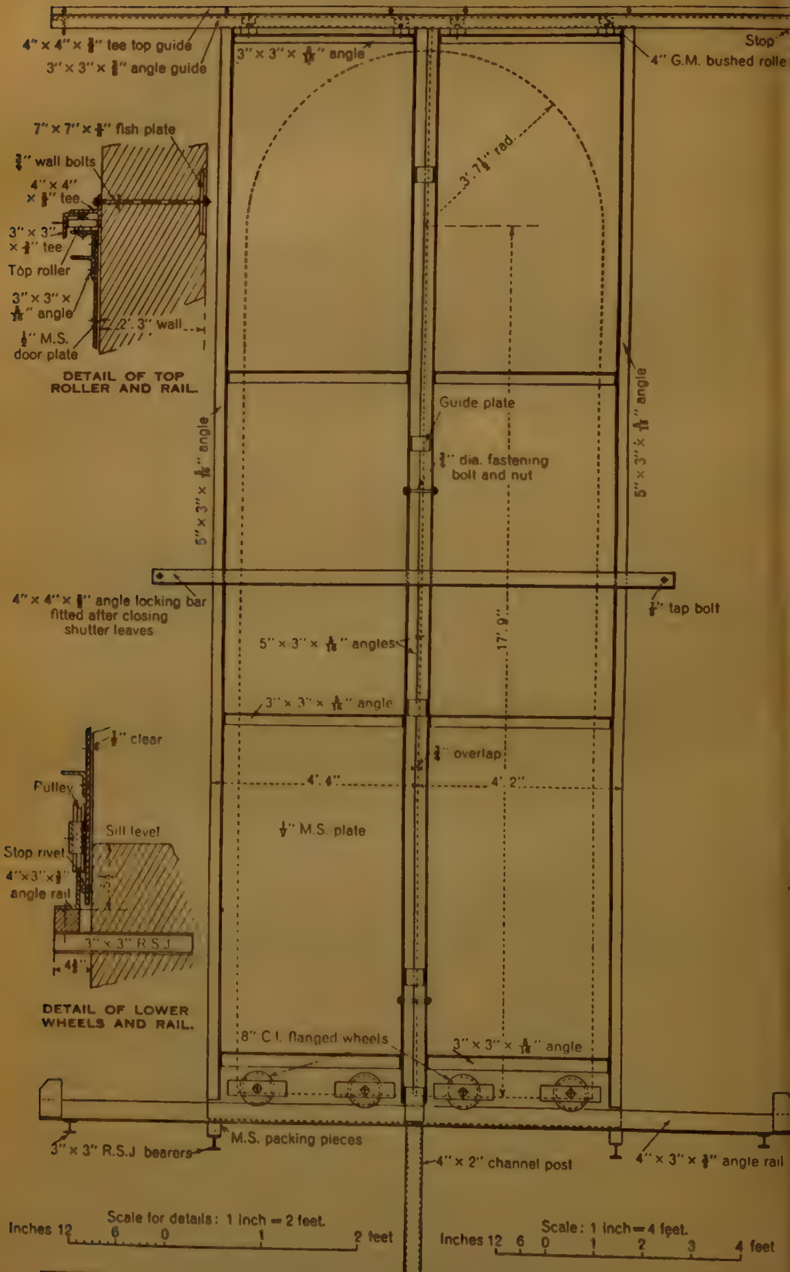
The shutters consist of easily-handled 500-lb. sections and are lifte on to rag-bolts, built from 8 inches to 10 inches into the masonry walls, an bolted into place section by section. These shutters are normally store

Fig. 2.



OUTSIDE-TYPE FIXED SHUTTERS.

Figs 3.



FRONT ELEVATION OF SHUTTERS.

INSIDE-TYPE MOVEABLE SHUTTERS.

below each window and would be erected and left in place over a period of hostilities.

Shutters provided for the inside of buildings are as shown in *Figs. 3.* These are more elaborate in design and mounted on wheels and rollers in order that they may be opened and closed at short notice.

Whilst they give adequate protection from suction blast-waves, and are, therefore, preferable to the outside type, they are also provided with guard-rails, as shown, to hold the shutters should they be blown inwards off the travelling rails.

These shutters are all of $\frac{1}{2}$-inch mild-steel plate, adequately braced and supported. Whilst they do not conform entirely with the Home Office requirements, which specify $1\frac{1}{2}$-inch plate, the weight of which would be prohibitive, splinters from a 500-lb. bomb exploded at 50 feet distance would only just penetrate through and would do little further damage. It should be noted that the station walls themselves are of massive dimensions, as otherwise it would serve no purpose to fit shutters on walls incapable of standing similar blast-loadings.

Standard-type shelters.

To provide protection for men working on open works-sites or in light constructional shops a standard type of shelter has been designed, as shown in Figs. 4, Plate 1.

These shelters are designed to withstand blast, fragmentation, and concussion from high-explosive bombs and incendiary bombs.

Protection from gas is not provided for by the installation of filter-plant, as it was considered that danger from gas is negligible because of the sparsely populated areas in which the majority of these works are sited.

Provision is made, however, for sealing off the ventilating openings, so that in certain circumstances the shelters could be occupied and sealed up for a few hours, depending upon the number of occupants.

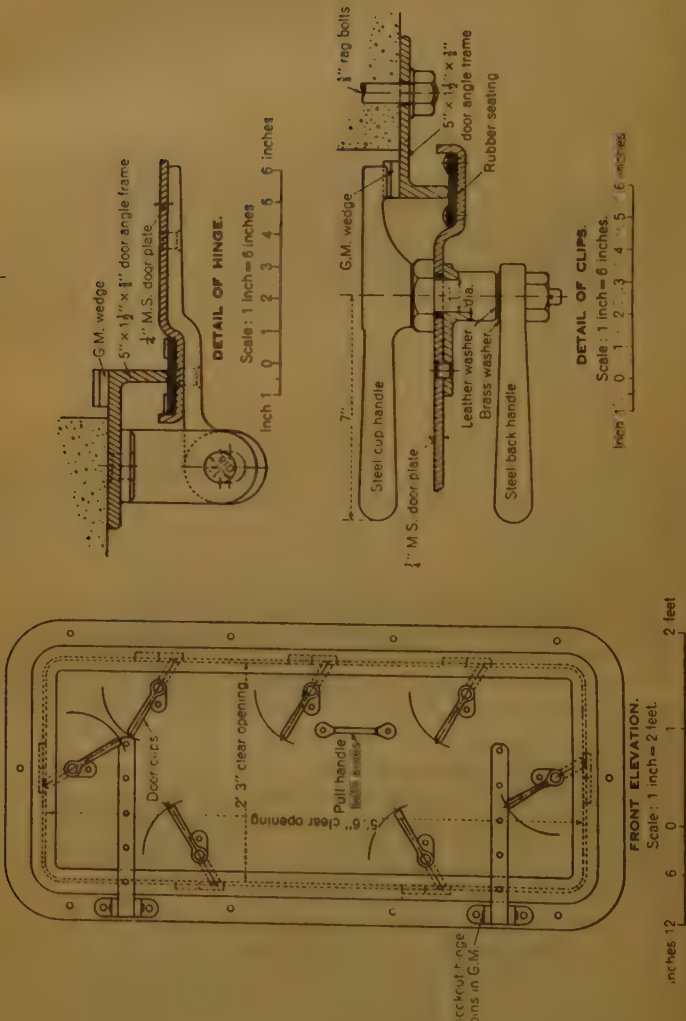
The shelter, as shown, provides accommodation for 50 men, and a similar type, but of half the length, is provided for accommodating 25 men.

Certain points in the design may be noted.

The entrance passages are so arranged as to provide a free passage for blast-waves, so that the doors are not subjected to the full blast pressure.

The doors are of particular note and are, therefore, shown in detail in *Figs. 5* (p. 578). It was decided that the doors can conceivably prove the weakest link in a shelter, as is generally found to be the case in shelters which have been built elsewhere or which are projected. The doors in question are adequate to withstand the adopted standards for blast and fragmentation, are gas-tight, and will stand a water-pressure of 10–20 lb. per square inch. They are of a type which has been used successfully on battleships where, in spite of severe buckling, they are generally capable of being levered off

Figs. 5.



WATER-TIGHT DOORS.

the frame. Their strength lies in the massive frame, the door closing on to the frame, and the adequate number of handles for wedging them tight.

As has been said above, this point of adequate door strength cannot be too strongly stressed, and the type has been adopted throughout all the shelters under review.

At first sight it might appear that better protection would be provided if the shelters were buried some 6–8 feet below ground-level. General site-conditions make this impossible as the subsoil water-level is, in most cases, only from 18 inches to 24 inches below the natural ground-level.

Further, the danger from flooding a low-level shelter is always present. Also, in the event of an earth-upheaval the work of clearing away the door-passages would be greater.

The shelters are self-contained, with water-supply from tanks, conveniences (chemical), and battery lighting (6-volt) for a 6-hour period. They are connected by telephone to the main buildings, and service-type gas-masks and decontamination clothing and equipment are also provided for certain of the occupants.

A group of shelters for a large office-building.

The group of shelters to be described below have a total capacity of 830 persons, and represent an adequate and extensive system of protection for office workers of one large building. It is applicable to any large city, and air-filtration is provided throughout.

The psychological effect on the type of persons who will use these shelters has been carefully considered, and the layouts, fittings, and interior finish have all been executed with a view to inspiring confidence and the provision of a reasonable degree of comfort and convenience.

The units consist of:

(1) A temporary outside shelter.
(2) A permanent basement shelter and control-room.
(3) A permanent culvert shelter.

Temporary outside shelter.

The term "temporary" is applied only in the sense that the location of this unit does not warrant its being considered as a permanent measure. The site is not an ideal one, being hemmed in between the outside wall of a four-storey building and a retaining wall backed by several feet of water. It was, however, the only cleared site available on which a start could be made right away, and, whilst designed and put in hand previously, construction was started during the recent crisis.

It will be noted that the design permits rapid erection of the shelter on any open site.

The general lay-out and details of construction are shown in Figs. Plate 1, and apart from the disadvantages of the site the design is adequa in itself. Provision has been made against the standards adopted fo blast, fragmentation, and concussion, and a further demolition factor 100 lb. per square foot over the whole area of the shelter has also bee provided. The doors are of standard type and the whole shelter is, there fore, proof against gas and flooding by water.

The shelter was designed to accommodate 120 persons and is, therefor provided with air filtration-plant.

Air is provided at a rate of 150 cubic feet per hour per person, th plant being operated electrically from the mains through a special trans former and switch-panel at 250 volts, or manually by two persons workin at one time.

The air is drawn in through a simple intake-pipe, protected, as far a possible, by a masonry chamber and concrete slab as shown. Apart fro protection it was not considered necessary to extend the pipe higher becaus of the low-lying areas in the vicinity to which persistent gas would tend t flow. The outlet-pipes are similarly protected, the exhaust-valves bein in the shelters.

The air-distribution is through ducting and outlets arranged suitabl at floor-level to give even distribution and to avoid draughts and noise.

25-volt lighting is also provided at approximately 5-foot candles, th whole being transformed from the 250-volt mains supply through th transformer and switch-panel, which throws the load automatically on to a battery of "Nife" accumulators, housed in the shelter, in the event o failure of the mains supply. The batteries are kept charged through a trickle-charger on the same panel during stand-by conditions, and have a 10-hour supply capacity.

Water-supply to several points is from a tank and the conveniences are all of the chemical type to avoid possible danger from gas leaking back along soil-pipes in the event of the water-traps being damaged.

The shelter is connected with a main control-room by telephone.

In operation, air-locks over each door are required, as shown in detail in Figs. 6, Plate 1. These are of the simplest possible type, consisting of heavy blanket material, battened to keep them up against a sealing strip on a sloping wooden frame. In spite of their simple construction they are most effective, especially if the blanket is wetted when in use. It will be remembered that the air filtration-plant must maintain a small positive pressure in the shelter, and the air-locks are, therefore, necessary if the doors have to be opened for persons passing in or out while the plant is in operation.

Permanent basement shelter.

The design and lay-out of this shelter is shown in Figs. 7, Plate 2. It

is an example of an entirely self-supporting structure located in a basement. The problem of strengthening the existing roof was considered in detail and a design prepared, but it was then decided that to build an entirely new structure, even in the limited space available, was a better solution.

The structure therefore takes the form of a mass-concrete arch, 15 inches thick at the crown and 24 inches thick at the springing levels, the abutment-walls being designed to take all the arch-thrust off the existing masonry side-walls. The arch is designed for a debris load of 350 lb. per square foot over the whole area, neglecting the steel reinforcement which has been carried through to bind the concrete against flaking on the underside, and which adds considerably to the strength of the arch.

Thus for a building with solid wall-construction, as in this case, there is a very high factor of safety provided, and because of this point and its situation in the building, it houses the control-room. This room will be connected with the outside General Post Office lines and also to the other shelters by inside telephones for the issuing of warnings, orders and instructions and for receipt of reports.

The shelter will accommodate 230 persons and is provided with suitable filter-plant, seating accommodation, conveniences, water-supply, alternative mains and battery lighting, telephones, etc., as for the temporary shelter.

In place of separate air-locks over each doorway an existing passage running parallel with the shelter has been rendered gas-tight. This has been accomplished by building two light cross walls at each end, with suitable door openings over which blanket-type air-locks have been constructed. The existing windows into the area alongside are fitted with frames which can be bolted into position with rubber sealing strips, thus making them gas-tight.

This extensive gas-tight passage has several advantages. In it stragglers and late-comers can be marshalled into groups to pass into the shelter at one time, thus avoiding frequent opening and closing of the shelter-doors, with resultant loss of filtered air.

Tools and equipment are also stored in it and the persons concerned with them can carry out their duties without having recourse to gas-masks. Further, it provides a very necessary line of protection against blast and concussion from a bomb exploding in the restricted area outside, and the same applies to the triangular spaces above the roof from bombs bursting above.

The ventilation pipes, of which there are six, are carried through to the outside area-wall and the intake-pipes run up to approximately 30 feet above the area floor-level. Two pipes form a duplicate air-intake system to the filter plant, and four outlets are available with non-return outlet-valves mounted inside the shelters.

Permanent culvert shelter.

This shelter, or rather system of shelters, has a capacity for 480 pers and is considered by the Author to be the best layout yet devised and offer the best protection.

The entire system is shown in Figs. 8, Plate 2.

It consists in part of remodelling and dividing up into three section disused brick culvert of fairly large section. Each section has a separ entrance, one by a stairway from ground-level and two through subw from two main shelter sections. The adjoining culvert sections interconnected through bulkhead doors of the standard type.

The two main shelters, adjacent to the culvert, are formed by the c struction of a 3-foot masonry wall, parallel to an existing massive brickw retaining wall. A 12-inch reinforced-concrete roof over the above wa forms a long chamber which is divided into two sections by a reinforc concrete cross wall with a bulkhead door in it. The main shelters r parallel to and alongside two sections of the culvert, and from each subway-entrance is provided to each culvert section. The main shelt are themselves reached by two subways leading off an area behind existing building, access to this area being provided by a broad flight steps down the earth ramp over the main shelters.

The essential feature of the whole layout is that each section is entire self-contained while being interconnected. Thus entrance and ex through other than the same door is possible by way of at least two se tions. Therefore in the event of any one section being demolished it do not cut off the exit from any other section.

As the soil round the culvert is partly waterlogged it was decided drain it off to ensure a dry inside surface for the culvert sections. T further drainage to waste of this seepage water and wash water from basin etc., offered some difficulties in maintaining the isolation of individu sections. The waste water is, therefore, run in the invert of the culve below a false floor. In each culvert section a sump is provided a normally the waste water is piped over these sumps to a common outfa into an existing soil drain. The piped lengths over the sump are, how ever, fitted with isolating valves and a branch with a valve for runnin into each sump. Thus should the drainage be obstructed in any o section the isolating valves can be closed while in the others the sum branch-valve can be opened and the water led into the sumps. The sumps provide several hours' capacity and would later have to be pumpe out.

Drainage from the main shelters offers no difficulties and is by way separate pipe-systems, the quantity to be drained coming entirely fro the wash basins and drinking fountains. Blow back of gas along thes pipes is prevented by having the traps sunk below the shelter-floors.

Chambers in reinforced concrete have been constructed at certai

points of the culvert to provide floor space and headroom for filtration plant, conveniences, etc. Space for blanket-type air-locks over each entrance door has been provided at the foot of the stairway entrance and in the two main shelter subways.

Each section is equipped with its own filtration unit with electrical or mechanical operation, dual mains and battery lighting, water tank, chemical conveniences, telephone, wash basins and drinking fountains.

Adequate earth cover is provided over all sections against blast and concussion and the three separate entrances ensure that the shelters are not completely cut off by demolition, earth-upheaval, or flooding. The roofs over the main shelters and the subways are further designed for an overload of 18 inches of debris over the whole shelter area.

The air inlets and outlets are similar in all respects to those provided for the temporary shelter.

Cost of Works.

In a review of costs it has been sought to reduce these to convenient units to enable them to be of use to others for estimating and comparative purposes.

The figures given below are for completed works ready for service or occupation:

(1) Fireproof catch-roofs	5*s*. 6*d*. per foot super, erected.
(2) Steel shutters for outside fitting . . .	6*s*. 3*d*. per foot super of window-area, erected.
(3) Steel shutters for inside fitting . . .	9*s*. 6*d*. ,,
(4) Standard-type shelter for 50 men . .	£550 complete (average of several sites).
(5) Standard-type shelter for 25 men . .	£400 ,,
(6) Temporary outside shelter for 120 persons	£1,150 ,,
(7) Basement shelter for 230 persons . .	£2,000 ,, (in "*Ciment Fondu*").
(8) Culvert shelter for 480 persons . . .	£4,500 ,,
(9) Standard-type bulkhead doors . . .	£17 10*s*. each for any number.

On a capacity basis the shelters cost from £8 to £10 per head. This figure compares very favourably with trench-type shelters, the costs of which run as follows:

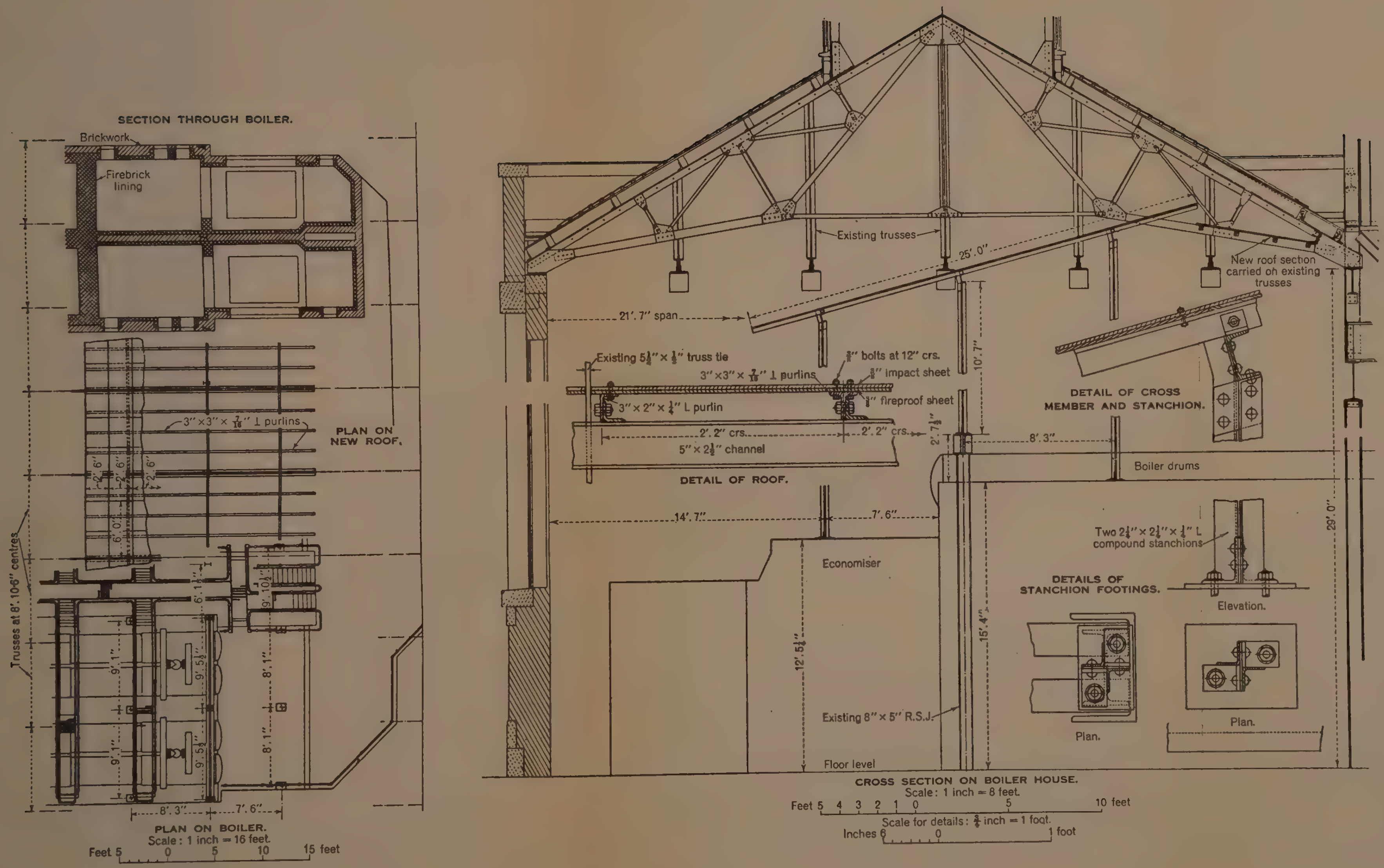
Open trenches	£2 per head.
Revetted timber sides	£3 17*s*. per head.
With roof of corrugated iron, etc. . .	£9 per head.

In conclusion, the Author hopes that the information and figures contained above will be of some material assistance to those engaged on similar works.

The works described are based entirely on his own interpretation of the design-data available at the present time. They are, therefore, open to criticism from any one wishing to indulge in speculation on actual conditions during a raid, of which the people, up to the present, have been spared any first-hand knowledge in Great Britain.

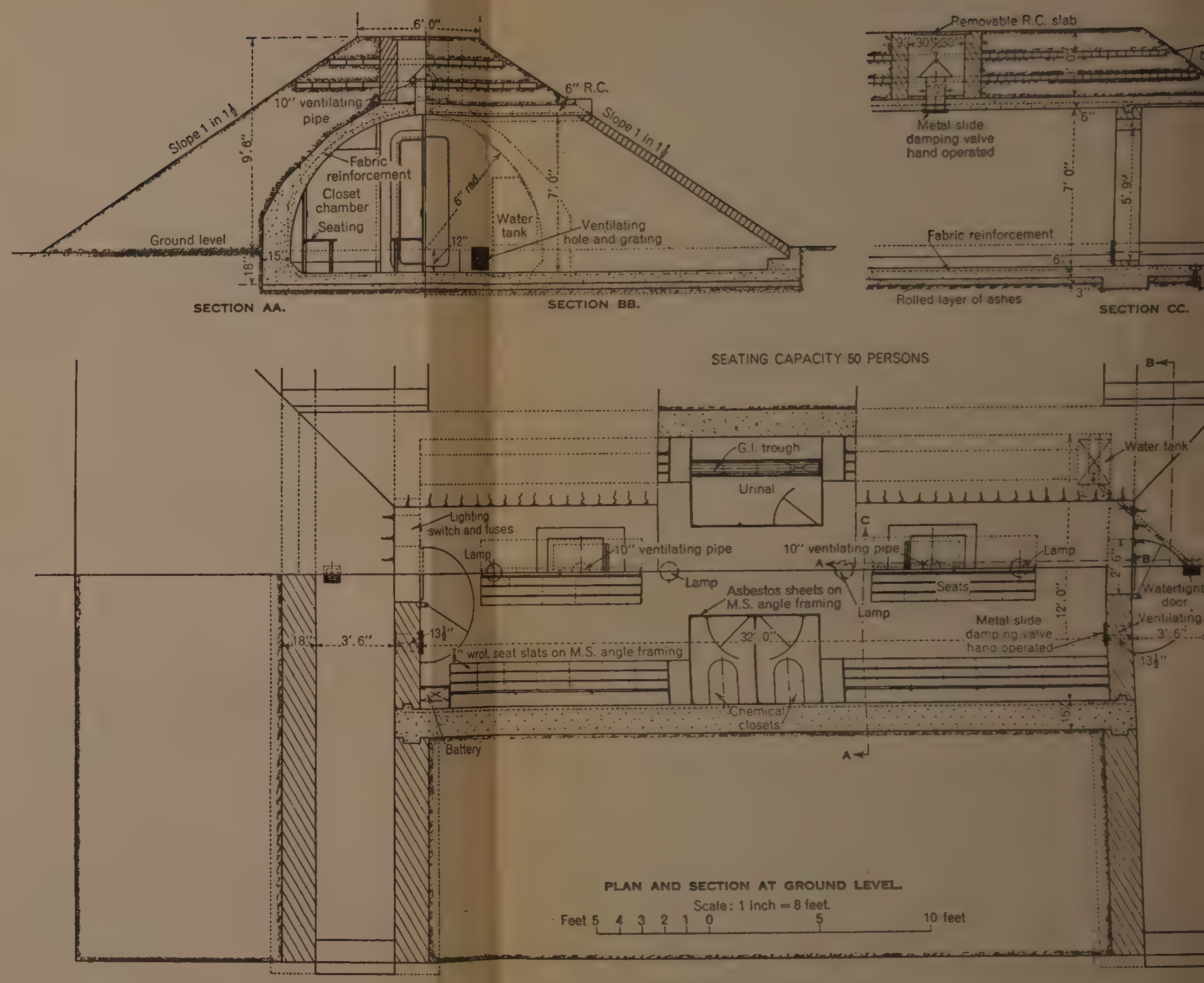
The Paper is accompanied by six sheets of drawings, from which Plates 1 and 2 and the Figures in the text have been prepared.

Figs: 1.



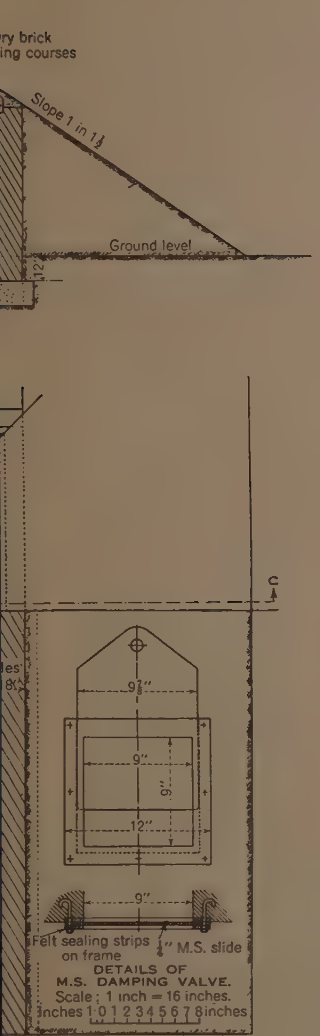
FIREPROOF CATCH-ROOF.

Figs: 4.

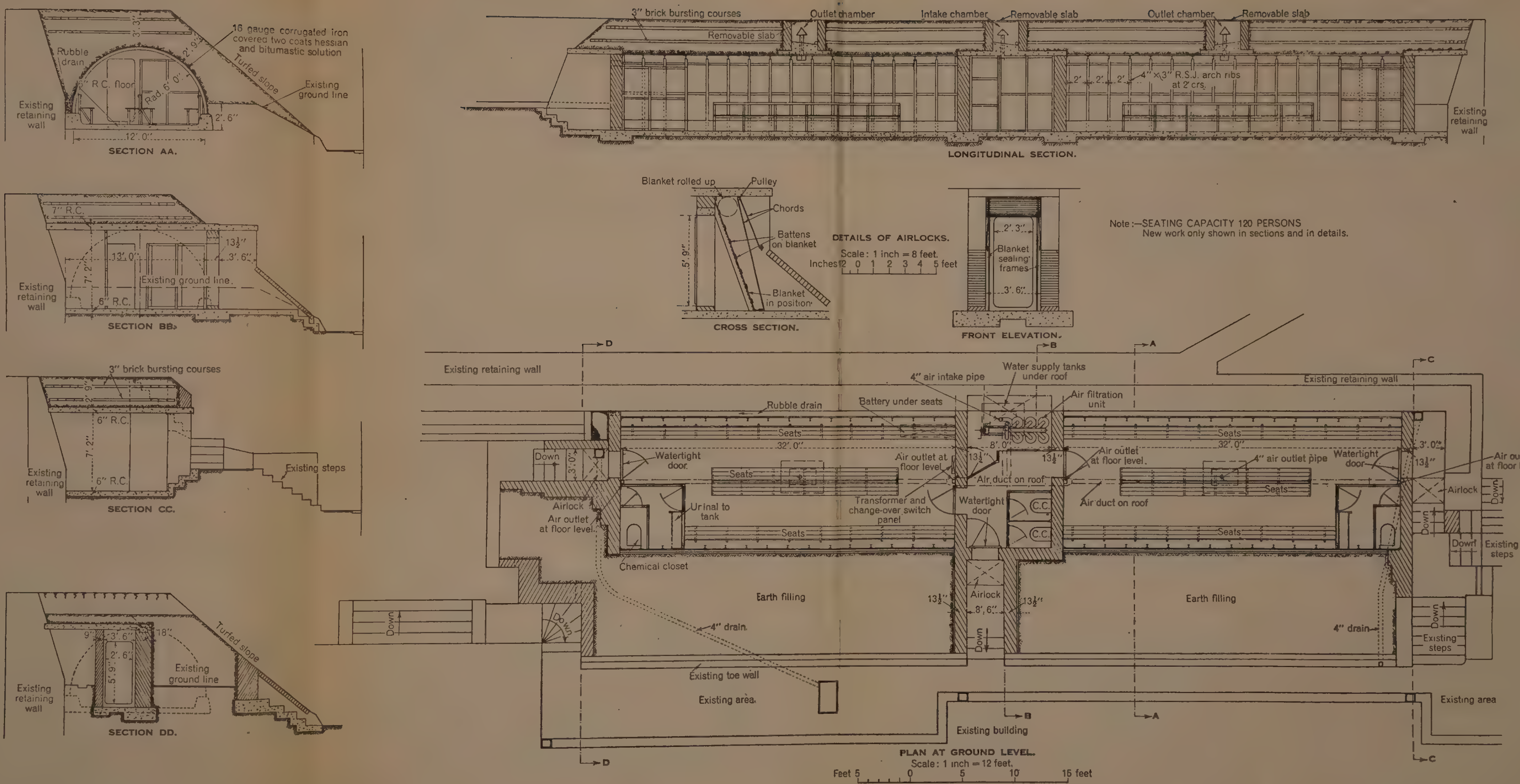


STANDARD-TYPE SHELTER.

William Clowes & Sons, Limited: London.



FIGS: 6.



TEMPORARY SHELTER.

R. W. G. CLERKE.

Paper No. 5206.

"Further Note on Flood-Hydrographs."

By Bertram Darell Richards, B.Sc. (Eng.), M. Inst. C.E.

(*Ordered by the Council to be published with written discussion.*)[1]

TABLE OF CONTENTS.

Introduction.

The present Paper is intended to be supplementary to the Author's Paper on "Flood-Hydrographs" already published by The Institution[2], and deals more particularly with the shape of flood-hydrographs and the causes affecting it. Certain modifications from the previous Paper are made in respect to the shape of the falling-flood curve and to the effect on the hydrograph of initial floods.

Flood problems generally fall under one of two categories:

1. The provision of flood passages.
2. Flood regulation.

In the former, the maximum intensity of the flood is the governing factor; in the latter the amount of run-off in a period is the important point, maximum intensity being of secondary interest, and the hydrograph is essential.

In the Paper referred to, the Author proposed formulas for the estimation of the maximum intensity and period of concentration of a flood from a catchment of given characteristics. These formulas form the basis of the hydrographs to be discussed, and are reproduced:

$$i = \frac{R \, . f(a)}{t+1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

[1] Correspondence on this Paper can be accepted until the 15th July, 1939, and will be published in the Institution Journal for October 1939.—Sec. Inst. C.E.

[2] Journal Inst. C.E., vol. 5 (1936–37), p. 405. (March 1937).

$$\frac{t^3}{t+1} = \frac{C \,.\, L^2}{K \,.\, s \,.\, R \,.\, f(a)} \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$Q_m = 1000\, K \,.\, i \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where t denotes the period of rise of the flood or period of concentration in hours,

Q_m ,, the maximum intensity of the flood in cusecs per thousand acres,

i ,, the average intensity of the rainfall over the catchment in inches per hour,

a ,, the area of the catchment in units of 1,000 acres,

L ,, the distance in miles to the furthest point of the catchment,

s ,, the coefficient of steepness or slope,

K ,, the run-off coefficient,

R ,, the rainfall coefficient,

and C is a coefficient.

Coefficient C.

As shown in the previous Paper, C may be taken as an inverse function of $(K \,.\, R)$ only without introducing any serious error. Values of C for different values of $(K \,.\, R)$ were given. Further calculations indicate that a slight modification of C for the higher values of $(K \,.\, R)$ will give more consistent results over a wide range of those other factors which affect it to a lesser extent. The values of C thus modified are found to approximate to

$$C = \frac{0{\cdot}032}{K \,.\, R},$$

which may be taken as sufficiently close for practical purposes.

Shape of the Hydrograph.

The basis of the formulas is the reduction of the rainfall causing the flood, to a hypothetical storm of area equal to that of the catchment, of uniform intensity i, and of duration T equal to t, the period of concentration of the flood. From this it follows that the maximum intensity of flood occurs when the whole catchment is contributing.

The maximum intensity $= Q_m = (K \,.\, i \,.\, a)$.
The total run-off $= (K \,.\, i \,.\, a \,.\, t) = Q_m \,.\, t$.

The curve of rising flood is given by :

$$Q_1 = Q_m \,.\, \frac{a_1}{a}$$

and

$$t_1 = t \sqrt[3]{\frac{r^2}{L^2}},$$

where Q_1 denotes the intensity of flood at time t_1,
a_1 „ the area of the part of the catchment producing Q_1,
r „ the radius of the arc intercepting the area a_1,
and t_1 „ the time to reach the intensity Q_1.

At the time the storm ceases, the catchment is theoretically covered with water of depth d, varying from $(K \,.\, i \,.\, t)$ at point O to zero at point A, in *Fig. 1.*

Fig. 1.

The water now begins to recede from A. If it is assumed that it moves from A to B a distance x in time t_1,

then $$t_1^3 = \frac{C \,.\, x^2}{K \,.\, i \,.\, s}.$$

At t_1, the area contributing to the flood is $(a - a_x)$, where a_x denotes the area intercepted by an arc of radius x, and the intensity of flood

$$Q_1 = Q_m\left(1 - \frac{a_x}{a}\right).$$

When $x = L$, $$Q_1 = 0,$$

and $$t_1^3 = \frac{C \,.\, L^2}{K \,.\, i \,.\, s} = t^3,$$

where t is the period of concentration. The period of rise and fall of the flood is thus the same, the total period being $2t$.

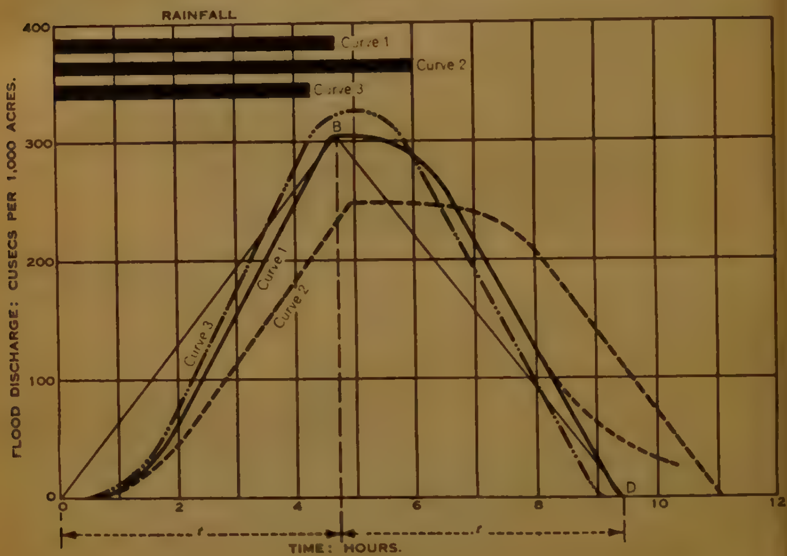
The hydrograph is given by Curve 1 in *Fig. 2* (p. 588). It will be noted that the rising curve is concave and that the area of the hydrograph for the period of rise is less than half the total area ; also that the falling curve is convex and is the reverse of the rising curve. The total area of the hydrograph is equal to the area of the triangle OBD, which equals $Q_m \,.\, t$, the total run-off.

This hydrograph is one for a rectangular catchment of area 40 square miles and with n, the ratio of length to breadth, equal to 1·67.

The shape of the catchment has obviously a great effect on that of the hydrograph, and as an illustration, a series of hydrographs is shown in *Fig. 3* (p. 589) for catchments of equal area and the same characteristics but of various shapes.

The coefficient K represents the coefficient of immediate run-off. Of the water lost, $(1 - K)$, a part may emerge later as ground-water, and if this occurs within the period of the flood, it will give rise to an indeterminate tail to the hydrograph as suggested by the dotted line on Curve 1

Fig. 2.



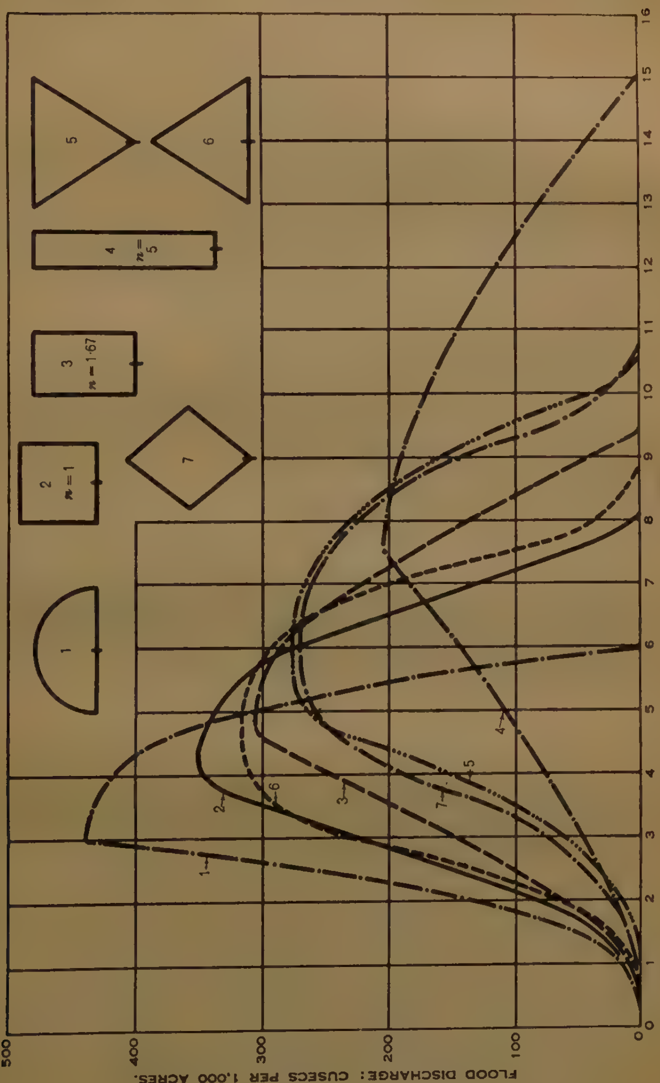
in *Fig. 2.* Part of the water might also be held in temporary storage, as in marshy areas, and, reappearing later in the flood, would splay out the curve of falling flood and increase the period of fall.

Variations from Conditions of Uniformity and Effect on the Hydrograph.

The above hydrograph is based upon certain assumptions of uniformity which are as follows :

(1). T, the duration of the flood, equals t, the period of concentration.
(2). i, the average intensity of the rainfall, is uniform over the whole catchment.
(3). i is uniform throughout the duration of the flood.
(4). K is uniform throughout the catchment.
(5). K is uniform throughout the duration of the flood.
(6). s, the coefficient of slope, is uniform throughout the catchment.

Fig. 3.



The effect on the hydrograph of variations from each of these conditions of uniformity will now be considered.

(1).—*T greater or less than t.*

Case (i) . . . $T > t$.

When T is greater than t, the equations become :

$$i = \frac{R \, . \, f(a)}{T+1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$t^3 = \frac{C \, . \, L^2}{K \, . \, i \, . \, s} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$Q_m = 1{,}000 \, K \, . \, i \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

The whole catchment will be contributing before the cessation of the storm. The flood will reach its maximum in time t, and remain at this for a peak period $(T - t)$, and will then fall in a further period t, in a curve which will be the reverse of that of the rising flood.

The hydrograph is given by Curve 2 in *Fig. 2*.

The case of $T > t$ is important in connexion with the limit of maximum flood-intensity for small catchments, considered on pp. 417–19 of the previous Paper[1].

Case (ii) . . . $T < t$.

Where T is less than t, $\qquad i = \dfrac{R \, . \, f(a)}{T+1}.$

The storm will cease before the water from the furthest part of the catchment has reached the point of concentration, and the area contributing at that time will be a_r, that intercepted by an arc of radius r, given by :

$$T^3 = \frac{C \, . \, r^2}{K \, . \, i \, . \, s} \quad \text{or} \quad r^2 = \frac{K \, . \, i \, . \, s \, . \, T^3}{C}.$$

Points on the curve of rising flood will be given by :

$$Q_1 = 1{,}000 \, K \, . \, i \, . \, \frac{a_1}{a}, \text{ up to the limit } a_1 = a_r,$$

and

$$t_1 = T \sqrt[3]{\frac{r_1^2}{r^2}}, \text{ up to the limit } r_1 = r.$$

At the time the storm ceases, the flood will have reached an intensity $Q = 1{,}000 \, K \, . \, i \, . \, \dfrac{a_r}{a}$, contributed by the area a_r. The catchment will theoretically be covered with water of depth d, varying from $(K \, . \, i \, . \, T)$ at point O, to zero at point A (*Fig. 1*).

Now assume that in time t_1 the water recedes from A to B, a distance x, the total time to recede to O being t.

The depth at B will fall from $\left(K \, . \, i. \, T \, . \, \dfrac{t_1}{t}\right)$ to zero.

[1] Footnote ([2]), p. 585.

The average velocity of flow from A to B will be two-thirds of the maximum velocity, so that

$$v_{av} = \frac{2}{3} c \sqrt{K \,.\, i \,.\, T \,.\, s \frac{t_1}{t}}.$$

Now $$t_1 = \frac{x}{v_{av}} = \frac{3x}{2c \sqrt{K \,.\, i \,.\, T \,.\, s \dfrac{t_1}{t}}},$$

whence $$t_1^3 = \frac{9x^2 t}{4c^2 \,.\, K \,.\, i \,.\, T \,.\, s},$$

and putting $$\frac{9}{4c^2} = C,$$

$$t_1^3 = \frac{Cx^2 t}{K \,.\, i \,.\, T \,.\, s}.$$

At t_1, $$Q_1 = 1{,}000\, K \,.\, i \,.\, \frac{(a_{r+x} - a_x)}{a},$$

with a limit of $r + x = L$ or $x = L - r$.

Q_1 will be a maximum when $x + r + x = L$, or $x = \dfrac{L - r}{2}$,

therefore $$Q_m = 1{,}000 Ki \frac{\left(a_{\frac{L+r}{2}} - a_{\frac{L-r}{2}}\right)}{a}.$$

At $x = L$, $t_1 = t$,

so $$t^3 = \frac{CL^2 t}{K \,.\, i \,.\, T \,.\, s} \quad \text{and since } T^3 = \frac{Cr^2}{K \,.\, i \,.\, s}.$$

Therefore $$t = T \,.\, \frac{L}{r}, \quad \text{or} \quad \frac{t}{T} = \frac{L}{r},$$

and therefore at x, $$t_1^3 = \frac{Cx^2}{K \,.\, i \,.\, s} \,.\, \frac{L}{r},$$

whence $$t_1 = \frac{T}{r} \sqrt[3]{Lx^2}.$$

At maximum flood Q_m, $x = \dfrac{L - r}{2}$,

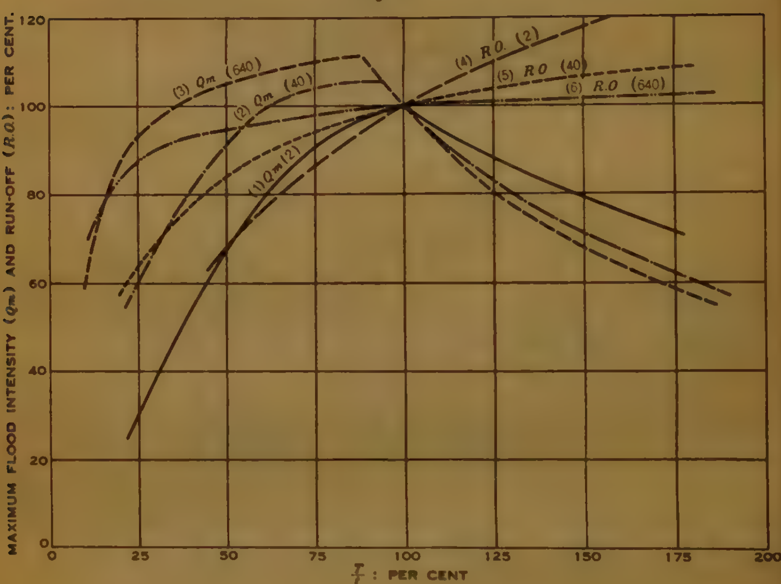
therefore t_1 at $Q_m = \dfrac{T}{r} \times \sqrt[3]{\dfrac{L(L - r)^2}{4}}$.

Hence :

$$\text{the total time of rise} = T\left(\frac{r + \sqrt[3]{\dfrac{L(L-r)^2}{4}}}{r}\right).$$

$$\text{and the time of fall} = T\left(\frac{L - \sqrt[3]{\dfrac{L(L-r)^2}{4}}}{r}\right)$$

Fig. 4.



Curves 1, 2, and 3 in *Fig. 2* show hydrographs for T equal to, greater than, and less than t respectively. The intensity and duration of the rainfall is also shown. As the ratio of T to t increases, the rise of the flood becomes slower.

To illustrate the effect of the variation of $\frac{T}{t}$ on the maximum flood-intensity (Q_m) and the total run-off, specific cases have been worked-out for small, medium and large catchments, rectangular in shape and with $n=1{\cdot}67$. Q_m, run-off, and $\frac{T}{t}$ have been put as 100 per cent. where $T=t$, and corresponding percentages have been worked out for other values of $\frac{T}{t}$, greater and less than unity. The results are shown in *Fig. 4*, where

curves (1), (2) and (3) show Q_m percentages plotted against $\frac{T}{t}$ percentages, and curves (4), (5) and (6) show run-off percentages plotted against $\frac{T}{t}$ percentages, for catchments of area 2, 40 and 640 square miles respectively.

The following points may be noted:

1. Q_m percentages increase more rapidly, and the maximum occurs at an earlier point, the larger the catchment-area.
2. Run-off percentage increases continuously with increase of $\frac{T}{t}$, and this increase is more rapid the larger the catchment-area until the 100 per cent. point is reached, and beyond that point is less rapid.
3. Where the Q_m percentage is a maximum, the run-off percentage is less than 100 per cent.

The maximum values of the Q_m percentage are 100, 106 and 110 per cent. for 2, 40, and 640 square miles respectively. A similar case has been worked out for a longer shaped rectangular catchment area with $n=5$, and in this case the Q_m percentage reaches a lower maximum. For 40 square miles it is 103 per cent. as against 106 per cent. when $n = 1{\cdot}67$.

It will thus be seen that ($T<t$) conditions may give a higher maximum flood-intensity than with $T = t$, and that this increases with the area of the catchment. With $T>t$, the maximum flood-intensity falls off rapidly. From the point of view of flood-passage provision, therefore, the condition of $T<t$ may be of importance, but even in the large catchment the increase is only 10 per cent.

Where Q_m is a maximum the run-off is less than 100 per cent. As the run-off increases the period of concentration increases, and the hydrograph, although it gains in area, becomes wider and lower. Where $T>t$, the percentage increase of run-off decreases with the catchment area.

The three hydrographs in *Fig. 2* are for a 40-square-mile catchment-area. The values of T are 4·75, 6·00, and 4·25 hours for Curves 1, 2, and 3 respectively, the last figure (4·25 hours) being that at which Q_m is a maximum. Flood-regulation has been worked out for each of these hydrographs for a reservoir area of 5 per cent. of the catchment area and a weir length of 30 feet per 1,000 acres of catchment. The maximum height of flood above the weir is found to be 1·36, 1·33, and 1·37 feet for hydrographs 1, 2, and 3 respectively, indicating that the variation of $\frac{T}{t}$ has not materially affected the regulation.

Incidentally, it may be noted that in using the short method of estimating reservoir lag-effect, described in Part 2 of the Author's previous Paper, the top of the hydrograph, where rounded, should be squared off. This adds little to its area.

As regards the effect of the variation of $\frac{T}{t}$, much depends on the actual shape of the catchment-area, but it would appear that generally speaking the assumption of $T=t$ does not lead to any serious under-estimate of flood-conditions.

2. *Variation of i over the catchment-area.*

If the intensity of the rainfall varies over the catchment-area, the condition which will produce the maximum flood-intensity is that in which the heaviest rainfall occurs near the point of concentration. The extreme case is therefore when a smaller storm of correspondingly greater intensity covers a part of the catchment only.

Let i denote the rainfall intensity of a storm covering the whole area a.
,, i_1 denote the rainfall intensity of a storm covering part area a_1.
,, t denote the period of concentration for a.
,, t_1 ,, ,, ,, ,, ,, ,, a_1.

Then
$$\frac{i_1}{i}=\frac{f(a_1)\,.\,(t+1)}{f(a)\,.\,(t_1+1)},$$

and the ratio of the flood-intensity from a_1 to that from a will be

$$100\,.\,\frac{i_1\,.\,a_1}{i\,.\,a}\text{ per cent.}=100\,.\,\frac{a_1f(a_1)(t+1)}{a\,.f(a)(t_1+1)}\text{ per cent.},$$

and the ratio of the total run-off from a_1 to that from a will be

$$100\,.\,\frac{i_1\,.\,a_1\,.\,t_1}{i\,.\,a\,.\,t}\text{ per cent.}=100\,.\,\frac{a_1f(a_1)\,.\,t_1(t+1)}{a\,.f(a)(t_1+1)t}\text{ per cent.}$$

In *Fig. 5* are shown curves of Q_m and run-off percentages plotted against a-percentage, for a rectangular catchment (with $n=1{\cdot}67$) of area 2 and 40 square miles respectively. The Q_m and run-off percentages both reach 100 per cent. when the whole catchment-area is exposed to the storm. The curves for the larger catchment have a more rapid rise than those for the smaller catchment. A much larger catchment area of 640 square miles has also been worked out and its curves approximately agree with those for 40 square miles.

The maximum intensity of flood would depend on the shape of the catchment-area. If there were a long narrow arm at the head of the catchment, as shown in *Fig. 6*, giving a small increase of area for a large increase of L, a higher flood-intensity would be found by omitting this arm in the calculation. Apart from such limitations, the maximum intensity of flood will be given by the whole catchment-area contributing, and hence by a condition of i uniform over the catchment. This applies to any size of catchment.

Fig. 5.

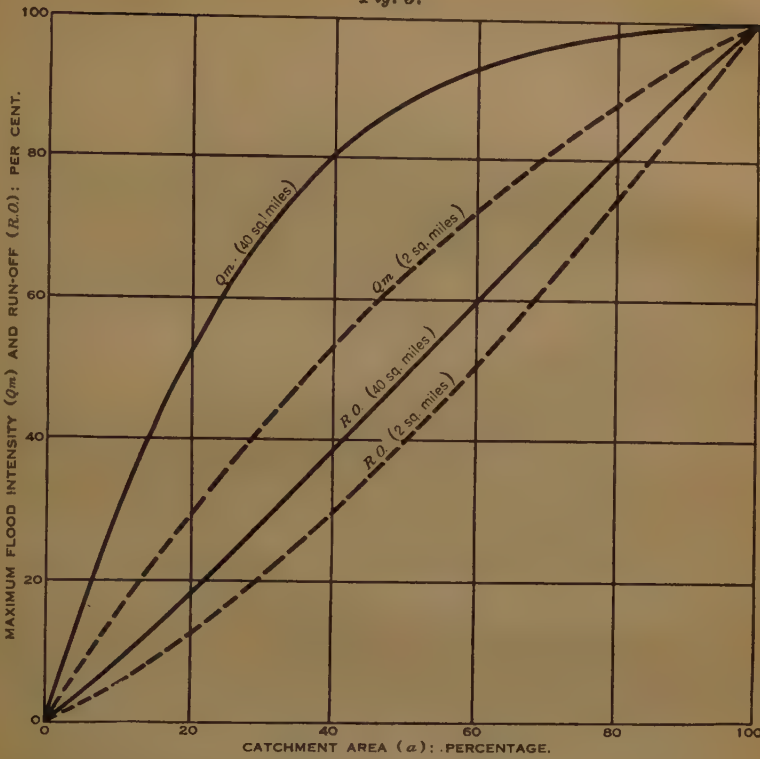
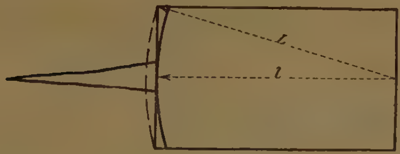


Fig. 6.



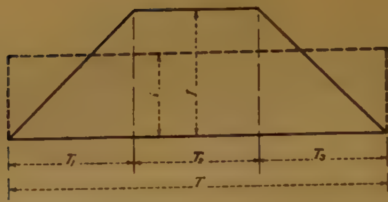
3. *Variation of i within the period of the storm.*

In the hydrograph considered, i has been treated as uniform within the period of the storm. In an actual storm, it would more probably rise and then fall off again. For the purpose of investigating the effect of variation of i, the rainfall-diagram has been considered as a trapezoid, as in *Fig.* 7 (p. 597).

The rainfall intensity rises from zero to a maximum I in time T_1, remains at I for period T_2, and then falls to zero again in period T_3.

The whole period of the storm $=(T_1+T_2+T_3)=T$, and T is, as before, taken as equal to the period of concentration t.

Fig. 7.



The average rainfall-intensity

$$i = I \, . \frac{T + T_2}{2T}.$$

By varying the ratios of $(T_1 : T_2 : T_3)$, a diagram can be obtained which would approximate to the actual rainfall diagram.

Let the water be reaching the point of concentration from a distance

L_1 at end of time T_1,
$L_1 + L_2$ „ „ „ T_2,
$L_1 + L_2 + L_3 = L$ „ „ „ T_3.

Treating each of the periods separately, the following equations are deduced :

$$T_1^3 = \frac{32}{9} \frac{C \, . \, L_1^2}{K \, . \, I \, . \, s} \quad \text{(I)}$$

$$\left(\frac{T_1}{2} + T_2\right)^{\frac{3}{2}} - \left(\frac{T_1}{2}\right)^{\frac{3}{2}} = \sqrt{\left(\frac{C \, . \, L_2^2}{K \, . \, I \, . \, s}\right)} \quad \text{(II)}$$

$$\frac{\alpha^2}{2}\left(\sin^{-1} \frac{T_3}{\alpha}\right) + \frac{T_3}{2}\sqrt{\alpha^2 - T_3^2} = \sqrt{\left(\frac{8}{9} \cdot \frac{C \, . \, L_3^2 \, . \, T_3}{K \, . \, I \, . \, s}\right)} \quad \text{(III)}$$

where $\alpha^2 = T_3(T + T_2)$.

Being given the ratios of T_1, T_2, and T_3 to T respectively, the above equations can be reduced to the form :

$$T^3 = A \, . \, L_1^2 = B \, . \, L_2^2 = C \, . \, L_3^2.$$

where A, B, and C are constants, and since $L_1 + L_2 + L_3 = L$ the value of each can be found as a fraction of L.

Taking equation (I) above and substituting for L_1 and T_1 the equivalent fractions of L and T, leads to an equation

$$T^3 = D \, . \frac{C \, . \, L^2}{K \, . \, I \, . \, s}, \text{ where } D \text{ is a constant.}$$

Now $$I = \frac{i \, . \, 2T}{T + T_2}, \quad \text{and} \quad i = \frac{R \, . f(a)}{T + 1},$$

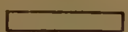
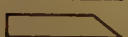
whence, substituting for I,

$$\frac{T^3}{T+1} = D_1 \cdot \frac{C \cdot L^2}{K \cdot S \cdot R \cdot f(a)}, \text{ where } D_1 \text{ is a constant,}$$

from which T can be found and thence i, and $Q_m = 1{,}000\ K \cdot i$. Similar equations to (I), (II), and (III) above, can be deduced for intermediate points on the curve of rising flood. The curve of falling flood will, as before, be the reverse of that of the rising flood, and the area of the hydrograph, or total run-off, will equal $Q_m \cdot T$.

The values of the maximum flood intensity, Q_m, and the run-off have been worked out for a number of rainfall diagrams based on variation of $T_1 : T_2 : T_3$. Q_m and run-off have been expressed as percentages of their values for the condition of i uniform. Two catchment areas of 40 and 2 square miles have been taken, with $n = 1{\cdot}67$, the characteristics of each being the same. The results are shown in Table I :—

Table I.

		Ratios of T.			40 square miles.		2 square miles.	
		T_1.	T_2.	T_3.	Q_m : per cent.	Run-off : per cent.	Q_m : per cent.	Run-off : per cent.
1		0	1·0	0	100	100	100	100
2		0·25	0·50	0·25	96·4	100	98·4	101·8
3		$0{\cdot}3\dot{3}$	$0{\cdot}3\dot{3}$	$0{\cdot}3\dot{3}$	96·4	100	98·4	101·8
4		0·50	0	0·50	95·4	100·3	97·8	102·1
5		0·0	0·75	0·25	104·6	98·6	102·5	97·9
6		0·0	0·50	0·50	107·8	97·7	104·5	96·1
7		0·0	0·25	0·75	111·1	97·1	106·9	94·4
8		0·0	0·0	1·0	113·0	96·9	107·1	93·9
9		0·50	0·50	0·0	85·9	102·6	91·9	107·2
10		1·0	0·0	0·0	79·7	103·8	87·8	110·8
11		0·50	0·25	0·25	90·8	101·4	95·1	104·3
12		0·25	0·25	0·50	102·6	98·9	101·5	98·9
13		$0{\cdot}6\dot{6}$	0·0	$0{\cdot}3\dot{3}$	88·8	101·6	94·3	105·1
14		$0{\cdot}3\dot{3}$	0·0	$0{\cdot}6\dot{6}$	102·0	99·0	101·6	98·9

The following points may be noted from Table I :—

1. When Q_m is a maximum, the run-off is a minimum, and conversely.
2. The range of variation of Q_m is greater in the larger catchment than in the smaller one.
3. The range of variation of run-off is greater in the small catchment than in the larger one.
4. Maximum Q_m and minimum run-off is given by the extreme case of the rainfall commencing at maximum intensity and gradually decreasing to zero, the maximum being twice the average intensity.
5. Minimum Q_m and maximum run-off is given by the other extreme of the rainfall increasing gradually from zero to maximum and then ceasing, the maximum being again twice the average intensity.

The maximum and minimum Q_m and run-off percentages have also been worked out for other similar catchments of different areas, and the combined results are shown in the following Table :

TABLE II.

Area : square miles.	Q_m : per cent.		Run-off : per cent.	
	Maximum.	Minimum.	Maximum.	Minimum.
2	107·1	87·8	110·8	93·9
40	113·0	79·7	103·8	96·9
100	114·6	78·0	102·7	98·2
640	115·6	76·0	101·2	98·9

It will be seen that the range of Q_m increases with the catchment area, while the range of run-off decreases.

As these maxima and minima are given by very extreme conditions of rainfall variation which would be unlikely to occur in nature, and in view of the comparatively small range of variation, it would appear that the assumption that i is uniform throughout the period of the storm would be unlikely to result in any serious underestimate of the flood-conditions.

4. *Variation of K over the catchment-area.*

This can only be treated by dividing the catchment-area into zones. Whilst this would complicate the calculations considerably, it seems unlikely that the results would be very different from those given by taking an average value of K.

5. *Variation of K within the period of the storm.*

K would tend to increase as the storm continued and the catchment-area became wetter. The effect would be to flatten out the lower part and steepen the upper part of the curve of rising flood. The falling-flood curve being the reverse of that of the rising flood, the net effect of the variation of K would be to give results little different from those based on an average value.

6. *Variation of s, the coefficient of slope.*

The slope has been taken as uniform throughout the catchment at an average value of s. Variations could be dealt with by dividing the catchment-area into zones. The general tendency would probably be for s to increase in the upper parts of the catchment, and the effect of this on the hydrograph would be to flatten out the lower part of the curve of rising flood and to steepen the upper part. The falling-flood curve being the reverse of that of the rising flood, the net effect of the variation of s would be to give results differing little from those obtained by assuming an average value.

Initial Floods.

At the time the storm commences there may already be a flow in the river arising from either:

(*a*) Lag-flow due to ground water.
(*b*) Initial flood due to continuous rainfall preceding the main storm.
(*c*) A previous storm, the flood from which has not run off when the main storm commences.

(*a*) *Lag-flow.*

Suppose that there is a continuous flow Q_o in the river, due to ground-water, when the storm occurs, giving a maximum flood intensity Q_m. The lag-flow will be superimposed upon this to give a total intensity of $Q_m + Q_o$. The normal hydrograph given by Curve 1 in *Fig. 8* (p. 600) will be raised bodily by an amount Q_o, giving that shown by Curve 2. The total run-off during the period of the flood will be increased by the lag-flow by an amount equal to $Q_o \times 2t$.

(*b*) *Initial Flood.*

Suppose that continuous but lighter rainfall has preceded the main storm and given rise to an initial flood Q_o. This may be considered as a separate storm caused by rainfall with some lower coefficient R_o. If t_o denotes the period of concentration of this initial flood,

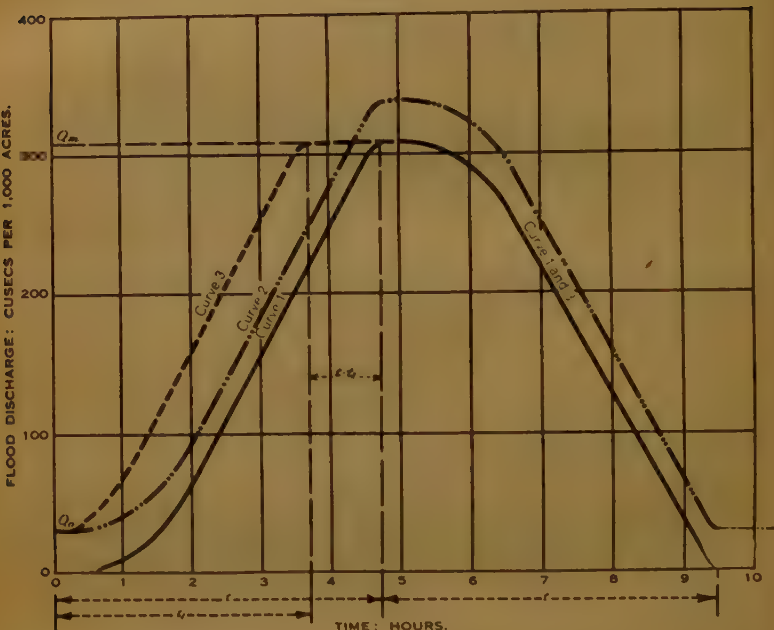
$$i_o = \frac{R_o f(a)}{t_o + 1}, \qquad (1)$$

and
$$\frac{t_o^3}{t_o+1}=\frac{C_o \,.\, L^2}{K \,.\, s \,.\, R_o \,.\, f(a)}, \qquad (2)$$

where C_o denotes the value of C corresponding to $K \,.\, R_o$

and
$$Q_o = 1{,}000\ K \,.\, i_o \qquad (3)$$

Fig. 8.



t_o, i_o and Q_o can be determined by assuming a value for R_o and using the corresponding value of C_o. Two or more values of R_o enable the value corresponding to the required value of Q_o to be found by interpolation, and hence i_o and t_o may be found.

Points on the rising-flood curve will be given by

$$Q_o' = Q_o \,.\, \frac{a_1}{a},$$

and
$$t_o' = t_o \sqrt[3]{\frac{r^2}{L^2}},$$

and the hydrograph can be drawn for the initial flood.

The area of the rising side of the hydrograph represents the water which has already run off when the major storm commences, and the area of the

falling side represents the water still to run off, which will be added to the flood from the major storm. When the major storm commences the rainfall will increase from i_o to i and will continue at this intensity for a period t. The maximum flood-intensity will be $Q_m = 1{,}000\,K\,.\,i$, but it will be reached in some time t_1 less than t and there will be a peak period $t - t_1$. t_1 can be determined as follows:

The velocity of flow from the catchment $= v = c\sqrt{ds}$. d will increase from $K(i_o\,.\,t_o)$ to $K(i_o\,.\,t_o + i\,.\,t_1)$.

$$\text{The average velocity } v_{av} = \frac{1}{t_1}\int_{t_1=0}^{t_1=t_1} v\,.\,dt_1$$

$$= \frac{1}{t_1}\int_{t_1=0}^{t_1=t_1} c\sqrt{K\,.\,s}\,\sqrt{b + it_1}\,.\,dt_1,$$

where $b = i_o t_o$,

$$= \frac{2c\sqrt{Ks}}{3it_1}\{(it_1 + b)^{\frac{3}{2}} - b^{\frac{3}{2}}\},$$

and since $t_1 = \dfrac{L}{v_{av}}$,

$$(it_1 + b)^{\frac{3}{2}} - b^{\frac{3}{2}} = \sqrt{\left(\frac{9\,.\,i^2\,.\,L^2}{4c^2\,.\,K\,.\,s}\right)} = \sqrt{\left(\frac{C\,.\,L^2 i^2}{K\,.\,s}\right)},$$

from which t_1 can be found.

For the co-ordinates of the curve of rising flood, it can be similarly deduced that:

$$(it_1 + b)^{\frac{3}{2}} - b^{\frac{3}{2}} = \sqrt{\left(\frac{C\,.\,r^2\,.\,i^2}{K\,.\,s}\right)}$$

and
$$Q_1 = Q_m\,.\,\frac{a_1}{a} + Q_o\,\frac{a - a_1}{a}$$

$$= Q_m\left(\frac{(N-1)a_1 + a}{N\,.\,a}\right),$$

where $\dfrac{Q_m}{Q_o} = N$.

The hydrograph of the combined flood can thus be drawn, as is given by Curve 3 in *Fig. 8*. The flood rises from Q_o to Q_m in time t_1, remains at Q_m for a period $(t - t_1)$, and then falls to zero in a further period t. The increase of the hydrograph area, that is, the area between Curves

1 and 3, represents the extra run-off due to the initial flood and is equal to the area of the falling side of the hydrograph of initial flood.

Approximate method.—It is seen that where Q_o is a small fraction of Q_m Curve 3 is practically parallel to Curve 1, although flattening out at the bottom to meet the axis at Q_o. This suggests a short approximate method of determining the hydrograph of the joint flood. The hydrograph for the main flood is drawn, the base being $2t$ and the height Q_m. This will also represent the hydrograph of the initial flood, by adjustment of the scales so that Q_m and $2t$ represent Q_o and $2t_o$ respectively. The area of the rising side of the hydrograph is determined as $\frac{1}{n}$th of the total area and the area of the falling side is therefore $\left(\frac{n-1}{n}\right)$ of the total.

The initial flood will then increase the area of the major flood-hydrograph by an amount

$$X = \frac{n-1}{n} . Q_o . t_o,$$

and, provided that Q_o is a fairly small fraction of Q_m, this increase will be approximately equal to $Q_m(t - t_1)$, whence the peak period $(t - t_1)$ $= \frac{n-1}{n} . \frac{Q_o}{Q_m} . t_o$ approximately.

Taking Curve 1, the hydrograph of the major flood, a horizontal line of length $(t - t_1)$ is drawn from the peak point towards the axis, and from the end of this line a curve is drawn parallel to Curve 1 and slightly flattened at the bottom to meet the axis at Q_o. Provided that Q_o is a small fraction of Q_m, say of the order of 10 per cent., this will give a fairly close approximation to the hydrograph of the joint flood. In the Author's previous Paper, the curve of rising flood for the joint hydrograph was drawn parallel to Curve 1 through a point Q_o on the axis. This, however, as will be seen, gives too large an area for the hydrograph.

It was suggested in the previous Paper that as regards the condition of wetness of the catchment prior to the flood, a wet catchment might be treated as equivalent to an initial flood, 10 per cent. of the maximum flood intensity being suggested as a reasonable allowance to make.

(c) *Consecutive storms.*

The third case of initial flow in the river is that in which the flood from a previous storm has not passed off when the major storm commences. Suppose that there are two consecutive storms with an interval t between them, and let t_1 and t_2 denote the periods of concentration of each respectively.

Then if $t \geqq t_1$, the first flood will have passed before the second storm commences.

$t < t_1$, the flood from the first storm will still be running when the second storm commences.

$t = 0$, initial flood conditions prevail and these have been discussed above.

The case to be considered is therefore the second. The flood from the first storm will continue running for a period $(t_1 - t)$ after the second storm has commenced, and provided that $(t_1 - t)$ is less than t_2, or in other words that $t > (t_1 - t_2)$, the first flood will have run off before the second storm has finished. The hydrograph for the joint storm can then be drawn by summing the ordinates of the two hydrographs at each point, as shown in *Fig. 9* (p. 604).

Comparison of Theoretical and Natural Hydrographs.

It will be seen from the foregoing what a diversity of factors affect the shape of the hydrograph. The theoretical hydrograph based upon assumptions of uniformity follows a definite type, however, as shown in *Fig. 3*.

The flood rises slowly at first, then more rapidly, with a flattening off towards the summit dependent on the shape of the catchment. The middle part of the curve is slightly concave in general. From the summit the flood falls off slowly at first, then more rapidly; the falling curve is the reverse of the rising curve and is generally slightly convex. The times of rise and fall are equal.

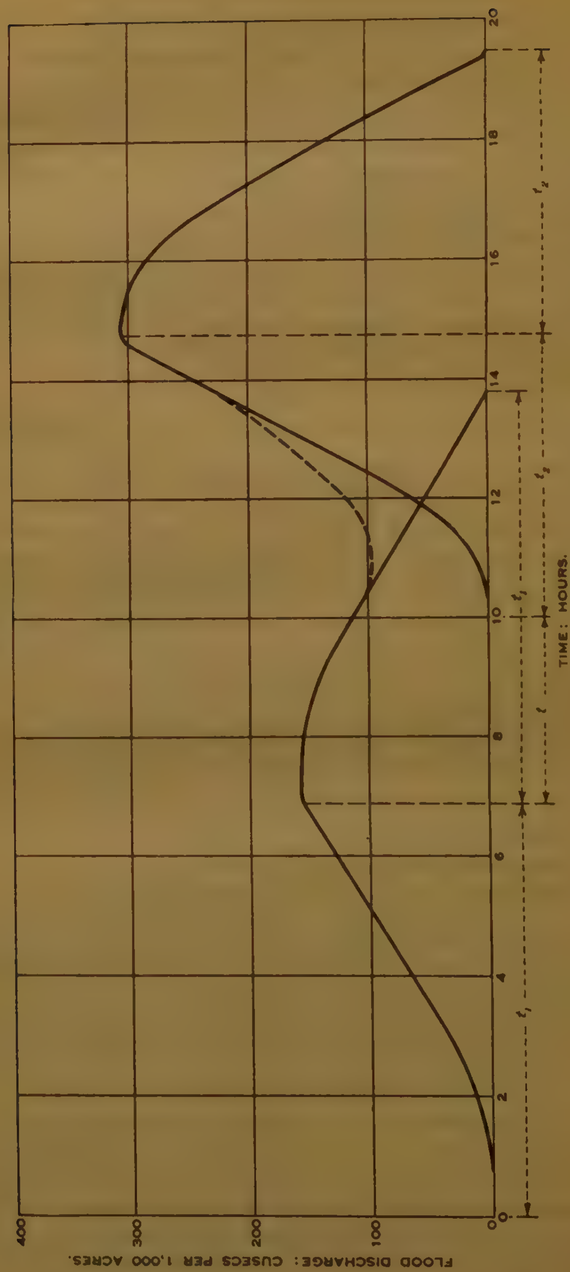
Natural hydrographs vary greatly, as might be expected, but the commonest type appears to have the following characteristics:

1. A slow preliminary rise, followed by a more rapid rise, generally in a slightly convex form.
2. A period of fall in excess of that of rise.
3. A considerable tail to the falling-flood curve.

Some natural hydrographs, however, show a concave curve of rising flood and a convex curve of falling flood; both types may, moreover, occur in different floods from the same catchment.

If the rainfall, instead of being uniform during the period of the storm, rose to a maximum and then fell off, as in case No. 4 of Table I (p. 597), the rising curve would tend to become convex in the middle. The same effect would be produced by a catchment very wide in the middle and narrow at the ends, as shown in *Fig. 3* (Curve 7). A peak period would be given either by an initial flood as in *Fig. 8* or by the duration of the storm being greater than the period of concentration, as shown in *Fig. 2* (Curve 2). Including this peak in the period of fall, the latter would be greater than the period of rise. The tail to the hydrograph would be given by groundwater or by temporary storage.

Fig. 9.


FLOOD DISCHARGE: CUSECS PER 1,000 ACRES.
TIME: HOURS.
0
2
4
6
8
10
12
14
16
18
20
100
200
300
400
t_1
t
t_2

Assuming that the areas of the theoretical and natural hydrographs are the same, the theoretical shape would tend to give more onerous conditions in respect to flood regulation than the natural one, and therefore to provide a margin of safety.

Conclusion.

The theoretical hydrograph is based upon certain assumptions regarding uniformity of conditions. Variations of some of these can be provided for, but this adds to the complexity of the calculations. Apart from this, however, there is the fundamental difficulty of establishing the data for precise calculations, and where very detailed data exist for a catchment it may be possible and expedient to determine the worst conditions of flood by analogy rather than by estimation.

The principal purpose of the formulas on which the theoretical hydrographs are based is to enable the conditions of maximum flood to be estimated from catchments for which the recorded data are limited. It has been shown that departures from the conditions of uniformity assumed are unlikely to lead to any important underestimate of the maximum flood, and still less of the total run-off. At the same time the theoretical hydrograph is more exacting in respect to flood regulation than the commonest type of natural hydrograph with its falling-flood curve splayed out, probably by temporary storage. For these reasons, the assumptions of uniformity which simplify the treatment appear to be justified, and moreover likely to give results which in respect to flood regulation should be on the safe side.

In using the formulas the coefficients should be assessed upon a conservative basis. The allowance of a 10-per-cent. initial flood to provide for the catchment being wet is desirable, although it might be inapplicable in the case of normally arid catchment areas subject to occasional storms. The actual shape of the catchment plays a very important part, and in fixing the length L it would be safer to ignore any long narrow arm at the head of the catchment.

There remains to be considered whether there is a limit to the area of the catchment to which the formulas are applicable, it being assumed that values can be assessed for the coefficients K, s, and R. It has been shown that:

1. Percentage increase of flood intensity with $T < t$ increases with the area of the catchment, but not to a very great extent. Percentage run-off only exceeds 100 per cent. when $T > t$, but the excess above 100 per cent. decreases with the catchment area.
2. The effect of variation of the rainfall-intensity over the area of the catchment is independent of the area.
3. The effect of the variation of the rainfall-intensity within the period of the storm increases with the catchment area in respect to

maximum intensity of flood, but decreases with respect to total run-off. The variation over a wide range of area is, however, not very large, even with extreme conditions of rainfall-variation.

It therefore appears that the formulas may be applied to large catchments. The maximum flood-intensity derived from them may be slightly too low, which would tend to a correspondingly low estimate of the flood-passage provision required; the total run-off would be slightly too high, which would tend to give a margin of safety in flood regulation.

Addendum: Moving Storms.

In the foregoing, only storms stationary over the catchment have been considered. It is now proposed to investigate briefly the effect on the flood-conditions when the storm moves down the catchment towards the point of concentration.

Assume a rectangular catchment of length l, width b, ratio of length to width n, and area a.

Assume that the storm has a length Nl and width b.

Then its area will equal $N \,.\, a$ and the ratio of length to breadth will be $N \,.\, n$.

Let this storm travel down the catchment towards the point of concentration.

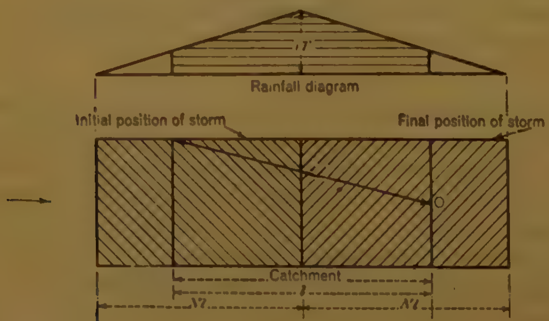
Case (i) . . . $N > 1$.

If N is greater than 1, the maximum flood-intensity will be less than with $N = 1$, since the intensity of the rainfall decreases with increase of area, while the distance to be travelled by the run-off to the point of concentration remains the same.

Case (ii) . . . $N = \frac{1}{2}$.

It can be shown that the largest flood occurs when the path of the storm is concentric with the catchment, as in *Figs. 10*, and its duration T

Figs. 10.

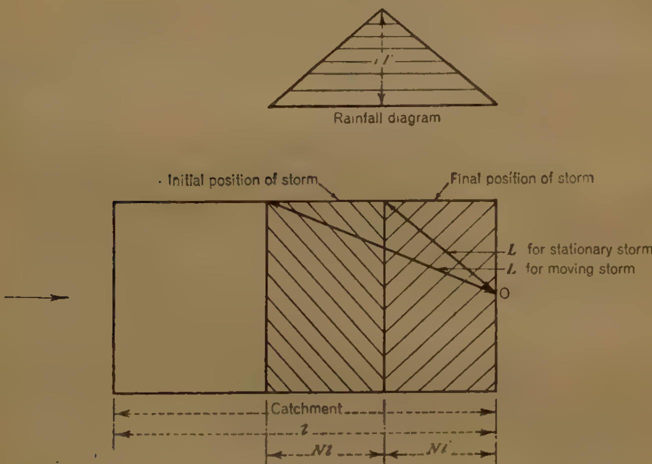


is equal to t, the period of concentration. The distribution of the rainfall in this event is also shown in the diagram.

Case (iii) . . . $N < \frac{1}{2}$.

The largest flood occurs when the storm is initially at a distance Nl from O, the point of concentration, and moves to B in time $T = t$, as shown in *Figs. 11.*

Figs. 11.



When N is less than $\frac{1}{2}$ all the rain will fall on the catchment, and when N equals $\frac{1}{2}$ the whole catchment will receive rain.

The comparative formulas for stationary and moving storms are given in the following Table:

TABLE III.

Storm.	N.	i.	$\frac{t^3}{t+1}$.	L^2.	ϕ.	Q.	Run-off.
Stationary	1 to 0	$\frac{Rf(Na)}{t+1}$	$\frac{CL^2}{K.S.R.f(Na)}$	$a\left(\frac{4N^2.n^2+1}{4n}\right)$	—	$N.K.a.i$	$Q.t$
Moving .	$> \frac{1}{2}$	,,	,,	$a\left(\frac{4n^2+1}{4n}\right)$	$1-\frac{1}{4N}$	$K.a.i$	$Q.t.\phi$
Moving .	$< \frac{1}{2}$	,,	,,	$a\left(\frac{16N^2.n^2+1}{4n}\right)$	$1-\frac{1}{4N}$	$2N.K.a.i$	$Q.t.\phi$

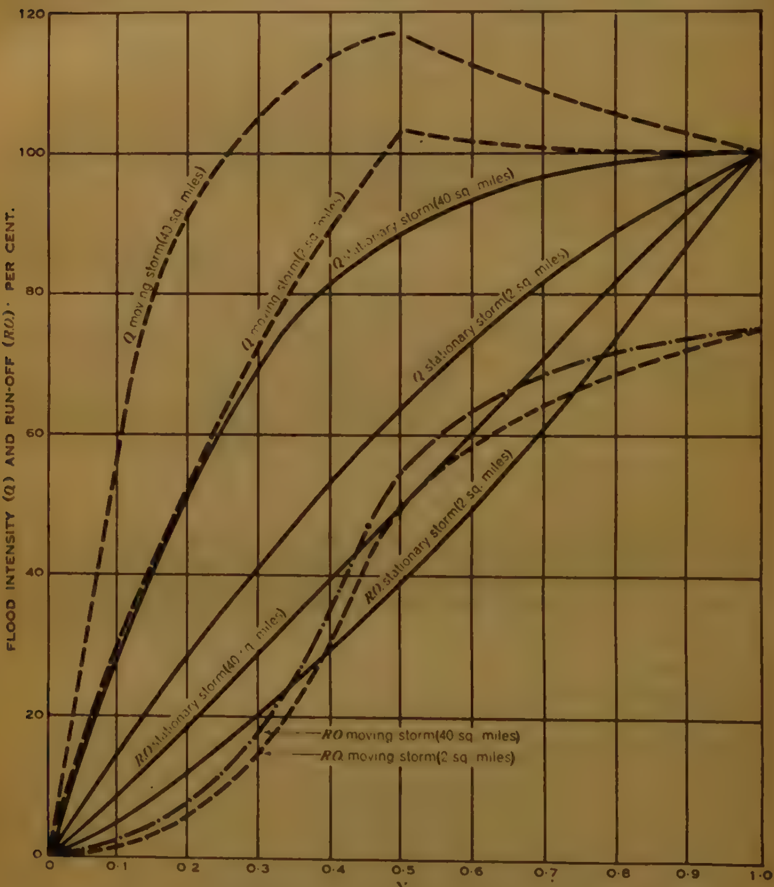
When $N = 1$, $\phi = \frac{3}{4}$ and when $N = \frac{1}{2}$, $\phi = \frac{1}{2}$.

It will be noted that a moving storm of area equal to that of the catch-

ment gives the same maximum flood-intensity, but only three-quarters of the run-off, as a stationary storm of the same area.

A moving storm of area equal to half that of the catchment is the smallest that will give rainfall over the whole catchment. The intensity

Fig. 12.



of the rainfall being an inverse function of the area of the storm, it follows that the maximum flood-intensity will be given by a storm of this area.

The flood-conditions of stationary and moving storms can be compared by taking a specific case. A rectangular catchment has been taken whose characteristics are as follows:

$n = 1{\cdot}67$, $R = 4$, $K = 0{\cdot}6$, $s = 0{\cdot}03$, and a has various values as shown.

The value of the function $f(Na)$ is taken from the curve given in *Fig. 1* (p. 407), of the Author's previous Paper[1].

The flood-intensity Q and the run-off from the catchment have been worked out for both stationary storms and moving storms for values of N from 0·1 to 1·0, and for catchment areas of from 2 to 640 square miles. Q and run-off are expressed as percentages of those for a stationary storm for each catchment, with $N = 1$. The results are shown graphically in *Fig. 12*, which gives the curves for stationary and moving storms on catchments of 2 and 40 square miles area.

It will be noted that in the case of the moving storm the Q-percentage rises to a maximum of over 100 per cent. at $N = 0{\cdot}5$, and then falls off slowly to 100 per cent. at $N = 1$, whilst the run-off percentage rises sharply up to $N = 0{\cdot}5$, and then more slowly to $N = 1$, when its value is 75 per cent.

For the stationary storm, the Q- and run-off percentages rise steadily to a maximum of 100 per cent. at $N = 1$.

The moving storm gives a greater Q percentage than the stationary storm of the same area for all values of N up to 1, when they become the same. The moving storm gives run-off percentages less than the stationary storm from $N = 0$ to about $N = 0{\cdot}40$, greater for $N =$ about 0·40 to about 0·70, and less again for N in excess of about 0·70 to $N = 1$, at which the moving storm gives 75 per cent. of the run-off for the stationary storm.

Maximum flood-intensity and run-off from moving storm.

The Q-percentage and the run-off percentage have been worked out for $N = 0{\cdot}5$, at which the Q-percentage is a maximum, for a number of catchments of the same characteristics and different area. In *Fig. 13* (p. 610) is plotted a curve of Q-percentages against the area of catchment. The excess over 100 per cent. reaches a maximum at an area of about 75 square miles, at which area the moving storm gives a flood-intensity 22 per cent. greater than a stationary storm covering the whole catchment. With larger areas this excess reduces rapidly, as $f(Na)$ becomes nearly constant. At 250 square miles and over, the moving storm gives practically the same flood-intensity as a stationary storm of the same area as the catchment.

On the same figure is shown the run-off percentage corresponding to the maximum value of Q. When Q is 122 per cent., the run-off percentage is 57 per cent.

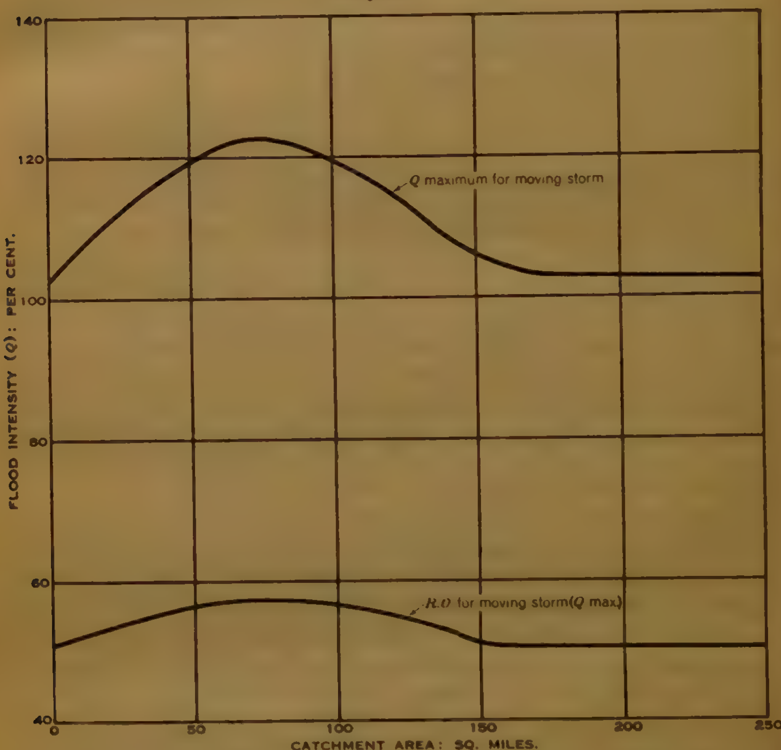
For very large areas $f(a)$ becomes constant at 0·60; at such limit the moving storm with $N = 0{\cdot}5$ gives Q at 100 per cent. and run-off at 50 per cent., while with $N = 1{\cdot}0$ it gives Q at 100 per cent. and run-off at 75 per cent.

It appears, therefore, that from the point of view of flood-passage provision, the moving storm gives more exacting conditions than the stationary storm for catchment areas up to say 150 square miles, beyond which the difference becomes unimportant.

[1] Footnote ([2]), p. 585.

The higher flood-intensity is, however, discounted by the reduced run-off. At an area of about 75 square miles, when the flood-intensity reaches a maximum of 122 per cent. of that of a stationary storm covering the catchment, the corresponding run-off is only 57 per cent. From the

Fig. 13.



point of view of flood-regulation, with which this Paper is primarily concerned, it would appear, generally speaking, safe to take a stationary storm covering the catchment as giving the most onerous conditions.

The results have been worked out for a narrower catchment, $n = 5$. The excess of flood-intensity given by the moving storm is found to be slightly lower, of the order of 3 per cent. lower at 75 square miles.

The Paper is accompanied by thirteen diagrams, from which the Figures in the text have been prepared.

Paper No. 5185.

"A New Theory of Turbulent Flow in Liquids of Small Viscosity."

By Thomas Blench, B.Sc., Assoc. M. Inst. C.E.

(Ordered by the Council to be published in Abstract form.) [1]

The Author employs modern physical ideas in the classification and analysis of authoritative flow formulas, and shows that they indicate the following universal flow formula for uniform turbulent conditions :—

$$U = \text{Absolute constant} \times \left(\frac{R}{x}\right)^{\frac{1}{4}} \times (gRS)^{\frac{1}{2}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where U denotes the mean velocity over a section during adequate time, R the hydraulic mean depth, S the slope (for a channel) or the non-dimensional pressure gradient (for a pipe), and x, whose dimension is that of length, measures the "brake" afforded by the boundary. When the boundary is "smooth," x is equal to δ, the thickness of the laminar film; when the boundary is "rough" it is the mean protuberance-height; and when the boundary is "incoherent" it is the equivalent protuberance.

In the first case the proper expression of R/δ leads to the generally accepted result of Blasius :—

$$U = \text{Absolute constant} \times N^{1/8} \times \sqrt{gRS}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

N being the Reynolds number UR/v.

In the second case the result, in its usual engineering form, is :—

$$U = \text{Constant} \times R^{\frac{2}{3}}\, S^{\frac{1}{2}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

which is also Manning's formula.

In the third case there results Gerald Lacey's flow formula for regime channels :—

$$U = \frac{\text{Constant}}{f^{\frac{1}{4}}} \times R^{\frac{3}{4}}\, S^{\frac{1}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

In fact, the particular formulas for the three types of boundary are derived from one general formula by proper expression of the term measuring the brake on the boundary. This is what would be expected from the fact that only one type of turbulence has been recognized.

[1] Copies of the Paper may be obtained on loan from the Loan Library of The Institution; a limited number of copies is also available, for retention by members, on application to the Secretary.

The next section of the Paper uses the type of argument employed by von Kármán, applied to the Author's improved model of turbulence, to deduce that, in a circular pipe, the velocity-distribution of the inner fluid from boundary to centre approaches a parabolic form. This section provides the link with von Kármán's work which, by assuming a turbulent model that is strictly applicable only to a gas, derives a logarithmic distribution-law inconsistent with the form of equation (1), although it fits the data very well. The velocity-distribution in an infinitely broad channel of uniform depth is also deduced and checked against experiment.

The last section of the Paper shows how the usual method of obtaining the Chezy equation $U=A\sqrt{RS}$ can be improved to give equation (1). It then proceeds to give to the generally understood term "boundary-velocity" a specific meaning in terms of a quadratic boundary-resistance law. Using this definition the energy-dissipation equation is written down and treated by the method underlying Hamilton's principle. The results are to find once more equation (1), to prove that the boundary-velocity is two-thirds of the mean velocity, to show that U^2/R is a factor in the measure of the mean force per lb. of fluid acting on the eddies which constitute turbulence, and to show that the rate of energy-dissipation is partitioned between the laminar film and the inner fluid in the ratio of 2 : 1. The more specialized results applicable to the Lacey theory are mentioned.

The definite meaning deduced from U^2/R justifies Lacey's inductive conclusion that U^2/R is a "turbulence criterion." The boundary-velocity ratio of $\frac{2}{3}$ is supported by authoritative experiment. The whole theory is of interest in comparison with the work of von Kármán, who deduces flow formulas by fitting a theoretical logarithmic curve to observed velocity-distributions. The Author deduces a universal flow formula from general dynamics, and obtains a parabolic distribution of velocity (in a circular pipe) by using von Kármán's methods and an improved model of turbulence.

The Paper is accompanied by six diagrams.

ENGINEERING RESEARCH.

THE INSTITUTION RESEARCH COMMITTEE.

Committee on Bituminous Jointing Materials for Concrete.

The name of Professor CYRIL BATHO, D.Sc., should be added to the list of members of this Committee published in the March issue of the Journal (p. 382).

RESEARCH IN THE ENGINEERING DEPARTMENT OF THE UNIVERSITY OF CAMBRIDGE, MARCH 1939.

THE following notes describe briefly some of the researches in progress or recently completed in the Department, under the general charge of Professor C. E. Inglis, O.B.E., M.A., F.R.S. (Professor of Engineering), and Professor B. Melvill Jones, A.F.C., M.A., F.R.S. (Professor of Aeronautics).

Much attention has been given to various problems of vibration; in particular, the vertical motion and the lateral oscillation of wheels and vehicles moving along a railway track have been investigated experimentally and analytically, the results having been published in the Institution Journal [1].

A preliminary investigation of the magnitude of the secondary stresses in a framed structure and of their effect on the stability of compression-members has been carried out, using a simple triangular frame, and it is hoped to extend this work by tests on a model Warren truss specially designed to ensure accuracy of construction and measurement; this work is being carried out on behalf of the Committee on Simply Supported Steel Bridges set up jointly by the Institutions of Civil and Structural Engineers.

The effect of the speed of testing on the properties shown by ferrous and non-ferrous materials is receiving special attention, the autographic tensile and torsion testing machines devised by Mr. H. Quinney having proved especially useful in that connexion. With ferrous materials the rate of application of strain has proved to be of great importance; for example, in tensile tests on Yorkshire iron the stress at the initial yield-

[1] Prof. C. E. Inglis, "The Vertical Path of a Wheel Moving along a Railway Track." Journal Inst. C.E., vol. 11 (1938–39), p. 262. (March 1939.)

Dr. R. D. Davies, "Some Experiments on the Lateral Oscillation of Railway Vehicles." *Ibid*, p. 224.

point was found to range from 23·9 tons per square inch (when the yield-point was reached in 0·2 second) to 13·2 tons per square inch (when the yield-point was reached in $1{\cdot}9 \times 10^6$ seconds). Mild steels and many alloy steels exhibit similar behaviour, and the ductility and the ultimate strength generally increase with increased speed of testing. Almost the only important exception is the behaviour of a water-quenched manganese steel (containing $13\frac{1}{2}$ per cent. Mn and $1\frac{1}{2}$ per cent. C) in which the initial yield-stress is decreased but the ultimate strength increased and the ductility more than doubled when tested at a high rate of straining. The yield-point strain in iron and steel is being investigated by Mr. A. M. Baxter.

A special machine has been developed for the high-speed testing of materials, the specimen being fractured by the release of a spring, and the energy absorbed by the fracture of the specimen being determined from a record of the initial deflexion and subsequent oscillation of the spring. The energy consumed in a fracture completed in $\frac{1}{1000}$ second has been found to be twice that for a slow-speed fracture of a similar specimen. The operation of the Riehle impact testing machine has been examined, and alternative methods of high-speed impact testing are being developed.

A comprehensive set of experiments has been performed on the progressive shortening of copper bars when longitudinal expansion due to temperature-rise is resisted. Various problems connected with the cold-working of metals, and, in particular, with the process of wire-drawing, have been studied in collaboration with Professor G. I. Taylor, F.R.S., who has also been associated with the above-mentioned work on testing.

The phenomena of ignition in compression-ignition oil engines are being studied in the heat-engine laboratory, where Dr. S. G. Bauer has been able to obtain satisfactory correlation of the ignition-lag in experimental explosion-vessels with that in actual engines. A spherical explosion-vessel has been employed, most of its volume being filled by two hemispherical electric heaters, between which a flat combustion-space is provided; the heaters can be rotated to induce turbulence. Pressures up to about 800 lb. per square inch and temperatures up to about 600° C. can be employed, and an indicator and thermocouples are provided. The engine employed is fitted with a special non-fouling quartz window, and the light emitted during combustion is reflected on to a small cylindrical steel mirror attached to the rim of the flywheel, the exact point of ignition being shown by the point in its travel at which the flywheel-mirror becomes illuminated. The engine has a quiescent type of combustion-head, and the compression-ratio, intake-pressure, injection-timing, and other characteristics are adjustable. A mechanical-optical high-speed indicator has been developed by Dr. Bauer [1], the essential element of which is a short twisted tube which tends to untwist when subjected to internal pressure.

[1] Described in *The Engineer*, vol. 167 (1938), p. 196. (19 August 1938.)

The free end of the tube carries a small mirror, the angular deflexion of which may be viewed by a synchronous rotating mirror or recorded photographically. For purposes of reference, the top dead centre is marked by a flash of very short duration from a mercury discharge valve.

Mr. P. de K. Dykes is working on the electro-magnetic recording of piston-ring flutter in internal-combustion engines, and Mr. A. Cruz de Sampaio is investigating the use of direct injection in petrol-engines.

A research has recently been completed by Mr. J. Diamond on the transfer of heat from air at high pressure to a steel pipe, under conditions of forced convection, and an investigation has now been commenced on the effect of temperature on the thermal conductivity of metals. Mr. T. S. Keeble is studying the heat-transmission from a current of hot air to finely-atomized liquid drops.

In the electrical laboratory, Mr. G. Wohlgemuth is investigating methods of contrast-expansion in sound-reproduction.

Two major researches in aeronautics are at present in progress. The fundamental mechanism of air-flow over a surface and the onset of turbulence in the boundary layer are being studied, both in the wind tunnel and in flight; this problem is of very great importance, as the most promising method of improving the efficiency and performance of aircraft appears to lie in the reduction of skin-friction. With this object in view, the motion in the boundary layer close to the surface is being investigated by hot-wire anemometers. In the earlier wind-tunnel tests it was found that turbulence commenced nearer to the leading edge of the surface than in corresponding tests in flight; this was shown to be due to micro-turbulence in the wind-tunnel air-stream (not present in free air), which has now been eliminated by an improved design of tunnel. In the laminar flow over the leading part of a surface in a wind tunnel, the anemometer shows small velocity-fluctuations with a period of the order of $\frac{1}{50}$ second; as the anemometer is moved towards the turbulent region, bursts of fluctuations with a period of the order of $\frac{1}{2000}$ second appear, and in the turbulent region these high-frequency fluctuations become continuous. Owing to difficulties caused by vibration in flying tests, it is not yet certain whether the fluctuations in the laminar flow are present in flight or whether they occur only in wind-tunnel tests. It is thought that the onset of turbulence may be connected with variations in the thickness of the laminar layer, but further experimental evidence is required upon this point. Although much work remains to be done in this field, it is notable that by special treatment of the leading edge of an aeroplane wing it has already been found possible to delay the point of onset of turbulence from 2 inches to about one-third of the chord behind the leading edge, with a corresponding improvement in performance.

The second aeronautical investigation at present in hand is the study of the response of an aeroplane to the forces exerted on its controls; for this purpose it is necessary to record simultaneously during flight twelve

independent measurements. Each of the dynamometers and accelerometers employed transmits its indication by varying the balance of a Wheatstone bridge, eleven galvanometers, an air-speed indicator and a stop-watch being arranged in a group and their indications recorded by a cinematograph camera. Each Wheatstone-bridge circuit is provided with controls for adjusting the sensitivity and setting the zero, and the observer is provided with check meters which can be plugged into individual circuits for observation independent of the photographic record.

The mechanical properties of plastic materials are being investigated by Dr. N. A. de Bruyne. Wood can be impregnated with synthetic resin to produce a material of very valuable properties, and laminated materials in which fabric of high tensile strength is bonded by resin have been produced with a tensile strength of 20 tons per square inch, a modulus of elasticity of $6{\cdot}5 \times 10^6$ lb. per square inch, and a density less than the lightest commercial magnesium alloy. The relative weakness of such material in shear may be overcome by moulding it in such a way that the grain of the material follows the direction of the principal stress in the finished article.

Mr. G. S. Gough, Mrs. G. H. Tipper and Dr. de Bruyne are investigating the stabilization of thin sheets in compression by means of a low-density continuous supporting medium. Apparently first suggested by William Fairbairn in 1848, it has recently been used to good effect in aircraft construction.

OBITUARY.

SIR HENRY FOWLER, K.B.E., D.Sc., was born at Evesham on the 29th July, 1870, and died at Spondon Hall, Derbyshire, on the 16th October, 1938. He was educated at Evesham Grammar School from 1879 to 1885, and then attended the Mason Science College, Birmingham, which was later incorporated in Birmingham University, from 1885 to 1887. His engineering apprenticeship was served in the Horwich works of the Lancashire and Yorkshire Railway from 1887 to 1891, and he was afterwards appointed assistant to the late Mr. George Hughes, then chief of the testing department on the same railway. Subsequently, he succeeded Mr. Hughes as chief of the testing department, and was also appointed gas engineer to the Lancashire and Yorkshire Railway. While at Horwich he attended the Railway Mechanics Institute, and was successful in 1891 in gaining the first Whitworth Exhibition awarded to a member of that Institute. He afterwards became a teacher in the same Institute.

In 1900 Fowler left Horwich to join the staff of the Midland Railway at Derby. He became in succession gas engineer, assistant works manager and works manager, and in 1910, after the retirement of Mr. R. M. Deeley, he was appointed Chief Mechanical Engineer to the Company. On the formation of the London Midland and Scottish Railway, he became deputy Chief Mechanical Engineer in 1923, and 2 years later was given the position of Chief Mechanical Engineer. His work in this high post will be chiefly remembered by the appearance of the "Royal Scot" class of locomotives, which were the most notable passenger engines of their day; but the reorganization of the whole system for the repair and overhaul of locomotives at Derby which led eventually to the possibility of greatly reducing the number of engines required for service was, in all probability, a service of still greater economic value. In 1931 he became Assistant to the Vice-President for Research and Development.

On the formation of the Ministry of Munitions in 1915 Fowler, who had been secretary to the Railway Companies' Munition Sub-Committee, was appointed Director of Production. In the following year he became superintendent of the Royal Aircraft Factory at Farnborough and in 1917 Assistant Director-General of Aircraft Production. He was Ministry of Munitions representative on the Aircraft Mission to the United States and Canada in 1918, chairman of the first Inter-Allied Conference on the Standardization of Aircraft Components, and Deputy Member of the Munitions Council in 1918–19. He also served on the Advisory Committee for Aeronautics, acting as chairman of its light alloys sub-committee. In 1917 he was created C.B.E., and in 1918 was advanced to K.B.E.

He was elected an Associate Member in 1896 and was transferred to the class of Member in 1918. He was a Member of Council from 1928–1934. In addition to being elected President of The Institution of Mechanical Engineers in 1927, he was also President of the following Institutions :—Institution of Locomotive Engineers in 1912, Institution of Automobile Engineers in 1920, Institution of Locomotive Inspectors and Foremen in 1921, and the Institute of Metals in 1932.

He was the James Forrest Lecturer in 1934, when the subject of his Lecture was "The Progress of Automobile Engineering." He also delivered in Session 1922–23 the Institution Lecture to Students on "Engineering Factory Organization."

He was a frequent contributor to the Proceedings of the various scientific and technical Institutions, and, in addition to obtaining a Miller Prize, as a Student, for his Paper on "The Testing and Inspection of Plates," he was awarded by The Institution a Telford Premium in 1897 for a Paper on "Calcium Carbide and Acetylene," a Watt Gold Medal and the Webb Prize in 1913 for a Paper on "Superheating Steam for Locomotives," and a Telford Gold Medal, jointly with Sir Nigel Gresley, in 1921, for their Paper on "Trials in Connection with the Vacuum Brake for Long Freight Trains."

He held the honorary degree of LL.D. from the University of Birmingham and of D.Sc. from the University of Manchester, and he was the first honorary graduate of the Manchester College of Technology. He was a Justice of the Peace, a Colonel in the Engineer and Railway Staff Corps, and the possessor of the Territorial Decoration.

In 1895 he married Emmie Needham, daughter of the late Mr. Philip Smith. She died in 1934. There were two sons and a daughter of the marriage.

SIR JOHN PURSER GRIFFITH was born on the 5th October, 1848, and died at Rathmines Castle, Dublin, on the 21st October, 1938. He attended Doctor Biggs' school at Devizes and Fulneck School at Leeds before going to Trinity College, Dublin, in 1865. After passing through the Engineering School in 1868 he started his pupilage with Dr. Bindon Blood Stoney, M. Inst. C.E. He then served for a short period as assistant surveyor for Co. Antrim in 1870, under Mr. Alexander Tate, afterwards being appointed assistant to Dr. Stoney in 1871, with whom he served for 28 years. He was appointed Chief Engineer to the Board in succession to Dr. Stoney in 1898, relinquishing this position in 1913. Throughout the whole of his association with the Dublin Port and Harbour Board, he was engaged on works of great magnitude, including the construction of roads, tramways, bridges, docks, quay walls, lighthouses, and stores. As an

example, the great concrete blocks used in the North Quay extension of the port of Dublin contained more than 5,000 cubic feet of masonry and weighed about 360 tons. These were built above high-water level, and when sufficiently set were lifted and transported by a floating shears or crane, and deposited on a bed provided by steam dredging, and levelled by men working in a large diving bell, entered through a tube fitted with an air-lock.

His engineering interests extended also beyond Dublin. He was advisory engineer to the Government on harbour and foreshore works at Wicklow and on Arklow Harbour, and he was a member of the Vice-Regal Commission on bridges over the Suir at Waterford and the Shannon at Portumna. He was also a member of the Royal Commission on Canals and Waterways appointed in 1906. He received a knighthood in 1911. In 1913 Sir John was elected a Commissioner of the Irish Lights—a board which has charge of the lighthouses, beacons, and buoys around the coast of Ireland. During the War he was Chairman of the Dublin Dockyard War Munitions Company.

After his retirement from the Dublin Docks and Harbour Board he was chairman of the Irish Peat Inquiry Committee in 1917–1918 and of the Water Power Resources of Ireland sub-committee appointed by the Board of Trade in 1918. He became a Senator of the Irish Free State in 1922. Late in his life he had the satisfaction of seeing his ideas for the utilization of the River Liffey come into favour, and his efforts were recognized by the Corporation of Dublin when, in 1936, it decided to confer the Freedom of the city upon him.

Sir John was elected Associate Member in 1877 and transferred to the class of Member in 1883. He was Member of Council from 1910 until he was elected Vice-President in 1916 and President in 1919. He was President in 1887 of the Institution of Civil Engineers in Ireland. At one time he was Vice-President of the Royal Dublin Society and member of the Royal Irish Academy.

He delivered the James Forrest Lecture in 1916 on "Development of Appliances for Handling Raw Materials and Merchandise at Ports and other Large Centres of Traffic." He also read a Paper on "Improvement of the Bar of Dublin Harbour by Artificial Scour" in 1878–1879, for which he was awarded a Manby Premium. During his year of Presidency, he presented the chandeliers and painting of the central roof panel, executed by Mr. Charles Sims, R.A., in the Great Hall of The Institution.

He married in 1871 Anna Benigna Fridlezius Burser. She died in 1912. They had two sons and a daughter.

DAVID HAY was born on the 10th April, 1859, and died at Flimwell Grange, Hawkhurst, Kent, on the 30th October, 1938. At the age of 18, he became a pupil, in 1877, of his father, and on the conclusion of his apprenticeship was appointed contractors' engineer upon the construction of the Great Northern and London and North-Western Joint Line from Newark to Tilton and Leicester, a total length of track of 44 miles. On the completion of that work he was employed, during 1884 and 1885, on the construction of a new dock at Silloth, Carlisle, after which he spent 3 years on the work involved in widening the North-Eastern Railway lines in and around Newcastle-on-Tyne. In 1888 he came to London to take up a position as contractors' engineer in connexion with the completion of the City and South London tube railway from the Elephant and Castle to King William street in the City, and on its extension to Stockwell. He was then for a short time in charge of the widening of the Great Northern main line near Grantham.

In 1892 Mr. Hay was appointed senior resident engineer, in charge of the construction of the Blackwall tunnel, for the London County Council. Soon afterwards Mr. Hay entered into partnership with the late Sir Benjamin Baker and Mr. (afterwards Sir) Basil Mott, the firm subsequently becoming Messrs. Mott, Hay and Anderson.

Among the many works with which Mr. Hay was subsequently associated was the reconstruction of the City and South London Railway during the years 1920–1924 and its extension in the 2 following years from Clapham Common to Morden, a distance of about 5 miles. He was also engaged upon the improvement of the Central London tube railway and upon the construction of thirteen bridges on the Liverpool-East Lancashire road.

He was elected an Associate Member in 1892 and was transferred to the class of Member in 1895. In 1897, in conjunction with the late Mr. (later Sir) Maurice Fitzmaurice he presented a Paper describing the construction of the Blackwall Tunnel, for which the Authors were awarded Watt Medals and Telford Premiums.

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